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OF THE
AMERICAN WATER WORKS ASSOCIATION

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Abstracts—News of the Field

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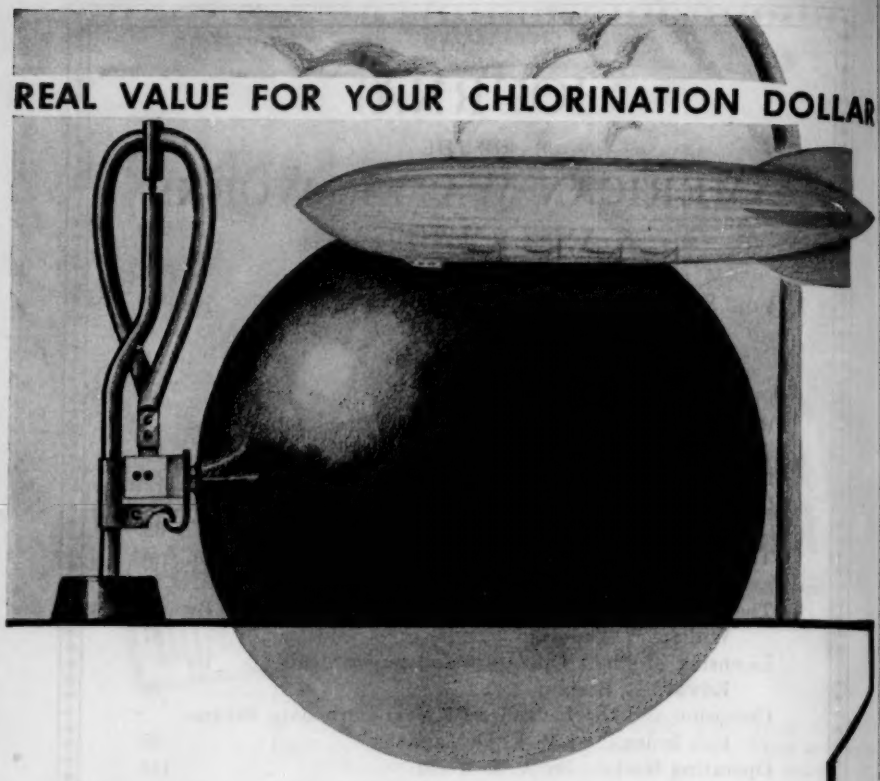

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Vol. 30 JANUARY, 1938 No. 1

FINANCING WATER MAIN EXTENSIONS*

BY L. A. JACKSON

(Manager, Municipal Water Works, Little Rock, Ark.)

Each month the water works manager eagerly studies his various operating reports. Special attention is given to those barometers of his business that denote expansion and growth. The manager is delighted if the statistics show an increase in net income, meters set, taps made, number of consumers and water sold. From year to year the average plant will show a normal increase in these items and this should reflect a very healthy condition.

But there is another side to the picture also, for these increases in business represent increases in fixed capital, and unless these capital expenditures are made on a financially sound basis, "growing pains" are bound to result.

In no branch of water works business is practice so varied as it is in the methods used to finance water main extensions. This probably is due, first to plain old 'difference of opinion'; and second, to the fact that this subject is more complex than it would first appear to be. It involves such related subjects as financial set up, rate structure, and legal requirements as they exist in the various cities.

For convenience, in my treatment of this subject all plant improve-

*Presented before the Southwest Section, October, 1937.

ments, betterments, extensions and additions are placed in two general classes.

1st—Major additions, or improvements that do not directly create new revenue.

2nd—Minor additions, or extension of mains that do create new revenue directly.

The separation is made in the above manner for the reason that most plants recognize the distinction and provide a different method of financing for each.

Four sources of money are commonly used for capital expenditures. These are:

1. Revenue Bonds
2. General Taxes or General Obligation Bonds
3. Assessments or Benefit Charges
4. Revenue from Water Rates

In general, major improvements, involving large sums of money, are financed by revenue bonds or general taxes.

The governing act in the State of Arkansas for financing improvements falling in this class has many desirable points. Some of these are:

The Act provides for the issuance of revenue bonds for the purchase, construction and improvement of a water works system.

A city ordinance must provide for their issuance.

The bonds shall bear interest at not more than 6 per cent.

A statutory mortgage lien shall exist upon the property.

The rates to be charged for service shall be sufficient to provide payment of interest upon all bonds, principal when due, expense for the operation and maintanance of the system and an adequate depreciation fund.

Rates for water must be fixed precedent to the issuance of bonds and shall not be reduced until the bonds have been fully paid, and may, if necessary, be increased in amount sufficient to provide for the payment of such bonds, both principle and interest, and to provide both proper funds for depreciation account, and operation and maintenance charges.

If any surplus shall be accumulated in the maintenance and operation fund, it may by act of Council, be transferred to

either the depreciation account or to the bond and interest redemption account.

Any municipality issuing revenue bonds shall install and maintain a proper system of accounts, showing amount of all revenue received and the application of same.

All of the funds received as income from a water works system and all funds received from the sale of revenue bonds issued for a water works system shall be kept apart from the other funds of the City.

The treasurer of the city shall be custodian of the funds, and shall be under proper bond, and shall maintain separate accounts for the various funds.

A separate ordinance may also provide for the execution by the municipality of an indenture defining the rights of the bondholders and this indenture, a Contract between the municipality and the Trustee, may by additional covenants, provide further safeguards for the handling of the various funds involved. These are only a few of the points in the Arkansas law.

Large capital expenditures are difficult to control even though the legal requirements are financially sound. Too often political influence and the uncertainties of the referendum either defeat a worthwhile project or spend more money than is necessary.

This paper, as the title implies, is not so much concerned with the financing of major improvements but rather with the problem that presents itself to the waterworks manager every day in establishing a course to follow in financing extensions of mains due to the normal demand to take care of new consumers.

How far should the water department go in making capital expenditures for new mains which create new revenue? Whether the department money is obtained from taxes, assessments or water rates, the financial plan used should be sound. I have examined a great many plans and even from this rather limited experience have learned that an extensive survey would probably reveal that no two are similar and many of them are unsound.

From a study of these various plans used it would seem that the prevailing practice is for the department to furnish a portion of the funds, the consumer to pay the balance, and the department to make refunds to the consumer.

The amount of money expended by the water department is usually established by capitalizing the anticipated or guaranteed revenue to be obtained from the new line on a gross per cent return basis. Some plants have a fixed requirement such as a per cent gross return that is considered sufficient by the management to warrant the capital expenditure.

These requirements are expressed in various ways; for example:

The annual gross revenue must be a certain per cent of the cost;
So many feet of pipe will be laid for each new consumer; or
Expend so much money per $10 of new revenue.

The refund practice also varies; for example;

Refund a certain multiple of the revenue received;
Refund a certain sum for each new service connection; or
Refund when the revenue from the line is so much per foot, etc.

The department requirements that I have had the privilege to analyze vary over a wide range, and it is my belief that many are set too low. (A digest of the answers to inquiry sent out and brief description of many plans in use is at the end of this paper.)

I talked to one operator about his extension financing practice, and he said he had a good plan. He said: "I spend $200, get $16 revenue per year and make 8 per cent on my investment." An analysis of this plan indicated that he was getting $16 per year per consumer and was furnishing the consumer 56,000 gallons of water per year. His operating expenses revealed that it cost him $12.32 to produce and deliver this 56,000 gallons of water. His net water profit therefore, was $3.68 per year or 1.84 per cent annual return. His bond service charges were 5½ per cent.

Before any fixed requirement is established all factors affecting the net return should be considered. Many managers lose sight of the fact that as their distribution system expands, their load and capacity demands increase and somewhere down the line there comes a day when a large capital expenditure is needed to bolster up the pumping plant, filter plant, or primary distribution system. This type of betterment usually creates no new revenue. Those little extensions that were installed two, five and ten years ago are blood relatives of this new project and each in proportion is responsible and should bear the burden of part of the expense. It is plain to see that unless these future necessary expenditures that cannot be compensated by new revenue are considered in some degree, a further reduction is reflected in the net return.

As a result of my correspondence with the many waterworks men who so generously assisted me in this brief survey, I have formed some general opinions which may be briefly stated as conclusions:

1. The extension project should be self liquidating.
2. Any financing plan used should have some degree of flexibility.
3. Final decision whether or not the expense is warranted should rest with the management.
4. The expense of the extension should be equitably distributed.
5. Proper records should be available to test the financial soundness of the plan in use.

It is customary after a study of this kind to advance some specific recommendations. Due to the confusion of ideas and variations in practice, I find this difficult to do. It is a fact however, that considerable progress has been made by our associated groups in establishing standards of good practice in other branches of our field. Therefore, it would seem reasonable that a concerted effort would produce similar results in perfecting a general policy or producing a manual on extension financing. This manual of procedure, to be of value, should be so constructed that it could be used as a guide in perfecting those plans now most generally in use.

DIGEST OF CURRENT METHODS

There follows a summary of the replies received from various cities. Whenever a percent gross return is indicated, it means that the department requires pre-contracts or agreements from prospective customers which will total such annual revenue as well as equal the given percentage of the cost of installing the main.

Columbus, Ohio. Assessment—deducting for street and alley intersections. City may assume varying amounts of the total cost. Ohio Law requires assessments be made according to benefits.

Bangor, Maine (6 per cent Gross Return). Consumer pays $6; per faucet per year until released from further payments by water board. Example: Main costing $1500 requires 15 faucets @ $6; which totals $90 or 6 per cent. The agreement was not clear whether this charge included water service.

Muscatine, Iowa. In some cases consumer pays part of the expense. Extensions must show a certain per cent in return.

St. Paul, Minnesota. Assessment of $1 per front foot of property served, collected at the rate of 10¢ per foot per year. No interest on deferred payments.

Denver, Colorado (Equivalent to 20 per cent Gross Return). Will extend 75 feet per consumer. Deposit required for cost in excess of 75 feet. Department refunds all water revenues for period of 5 years or refunds the cost of 75 feet each time main is extended 75 feet for new consumer during 5 years.

Parkersburg, W. Va. Issues bonds payable out of operating revenues. Act of Legislature gives this authority without vote of people.

Kalamazoo, Mich. (20 per cent Gross Return). Assessment of 75¢ per front foot (6 inch Main) payable in 5 annual installments; 5 per cent interest charged on deferred payments. When property becomes water user, refund not over $60 or 5 x gross revenue.

Louisville, Kentucky (15 per cent Gross Return). Consumer deposits entire cost (labor, material plus 15 per cent administrative charge). If revenue for any 12 consecutive months is equivalent to 15 per cent of cost within 5 year period total refund is made. Outside city limits same plan except refunds are based on new service connections.

Jacksonville, Florida. Extensive improvements financed through bonds. Ordinary extensions made from earnings.

Pueblo, Colorado. Extensive improvements financed by city bonds redeemed by special tax. Department pays for extension within city limits. Outside city limits revenue must justify any department expense.

Pasadena, California. Assessment 50¢ per front foot on 6-inch main basis or $1.00 per running foot. In new subdivisions and in streets outside city limits property is assessed cost of 6-inch cast iron main.

Kitchener, Ontario. Feeder mains financed from revenue or borrowed funds. Assessment 6–8¢ per frontage foot for 20 years on 6-inch main basis. Prior to 1920, consumers had to guarantee 10 per cent gross return.

Ashville, N. Carolina. Extensions financed by bond issues prior to 1930. PWA has done work past few years.

Sedalia, Mo. (12 to 15 per cent Gross Return).

Jackson, Miss. Revenue and issuance of bonds.

Cedar Rapids, Iowa (Admits plans are not satisfactory). Act provides assessments but states further that consumer may have water without charge for full amount of assessment within 15 years. Not satisfactory for small extension. Another Plan: Consumer pays cost—total refund when revenue for 12 months period equals 10 per cent of cost. After 7 years total refund is made in any event.

Austin, Texas (20 per cent Gross Return). Consumer pays entire cost and receives refund 5 times annual revenue—limit 10 years.

Cambridge, Mass. Financed by water revenues. State law provides that serial bonds can be issued for laying water mains 6-inches to 14-inches; 15 year term bonds 16-inches and greater; 20 year term bonds-standpipes, reservoirs, pump stations; 25 year term bonds filter plants.

Wilmington, Delaware (6 percent Gross Return). Financed from capital account fund as set up in budget; department stands the expense and is reimbursed annually at the rate of 6 per cent until revenue is sufficient to carry investment. Rural extensions should be self-supporting even if consumer pays entire cost.

Danvers, Mass. (10 per cent Gross Return). Department pays cost and is reimbursed 10 per cent for 5 years. Consider reducing this rate to encourage building development. Trend in east is to have a guarantee of 4 to 6 per cent.

Salt Lake City, Utah. City has what is known as refunding ordinance—consumer may borrow amount from city banks to pay for extensions. Principal (no interest) is refunded in 4 annual payments.

Spokane, Wash. Extensions financed out of revenue.

Concord, N. H. (Gross Return 8 per cent). Mains financed by revenue and bond issue; occasionally department requires gross return of 8 per cent.

Tallahassee, Fla. Financed from revenues set up in budget.

Phoenix, Arizona. Within city limits, mains are financed from revenue set up in budget; outside city limits, financed by consumer or realtor. Refund the first year's revenue from each consumer—10 year limit.

Pocatello, Idaho. State laws do not permit issuance of revenue bonds. Purchase of plant, extensions and improvements financed by general bonds paid by a general property tax levy.

Weehawken, N. J. (Hackensack Water Co.) Private Company (15 per cent gross return). If 15 per cent return is in sight company pays for extension. 1—If revenue is less than 50 per cent of that required; consumer advances cost of extension. Refunds are made, equal to estimated annual revenue for $3\frac{1}{2}$ years for each new service connection (10 yrs.) ($3\frac{1}{2} \times$ annual revenue). When line shows 15 per cent return, deposit is refunded. 2—If 50 per cent or more revenue is in sight: Applicant guarantees 15 per cent annual return

making up any deficiency and is under bond. Cancelled if revenue for 3 year period equals or exceeds guarantee—10 year limit.

Portland, Oregon (6 per cent gross return). Department makes extensions if line shows 6 per cent return. Installs 2-inch mains in outlying districts. *In some* instances consumer pays for lines 6-inches or 8-inches laid under department supervision and department purchases lines when same show 6 per cent return at cost less depreciation.

Watertown, Mass. Extension must be warranted in opinion of Department. Department pays cost.

Eugene, Oregon. Early years financed by issuance of bonds. Extensions now must be self-supporting.

Atlanta, Georgia. Appropriations from city general funds by act of council or bond issue. Extensions outside city limits paid for by property benefited.

Lincoln, Nebraska. Feeder mains paid for by department; Service mains financed by assessments. In some instances in large districts, entire cost including feeder mains assessed against abutting property.

Oak Park, Illinois. Special assessments.

Springfield, Illinois. 1—Special Assessment Plan—Assessments payable in cash or 3 yearly installments. 2—Consumer pays cost of extension and Department refunds equivalent of cost of laying 35 feet of main for each new consumer or when annual revenue equals 20¢ per running foot. (If main cost $2 per foot this means 10 per cent gross return.) 3—Extend 35 feet of main per consumer.

Madison, Wisconsin. Transmission mains are financed from surplus or water revenue bonds; service mains financed by assessments and part by department funds. State law allows assessments not to exceed ½ cost of 6-inch main. Experience in Madison indicates that water funds have accounted for ⅔ of cost of all extensions made and special assessments ⅓.

Harrisburg, Pa. Assessments $1.00 per foot frontage and if paid within 3 months entitle consumer to abatement of water rents equal to amount of assessment.

Baltimore, Maryland. Any unusual construction cost, surplus excavation, excess backfill etc., paid by consumer and not refundable. Consumer deposits entire cost—$1.50 per foot 6 inches or less. Department refunds $100 per new consumer per $20 of new revenue per year—10 year limit.

Burlington, Vermont (No Bonded Indebtedness). Department pays for extensions out of earnings.

Mobile, Ala. Issuance of bonds.

Charleston, S. C. (6-inch main). Consumer advances cost of line on 6-inch basis. Term of contract usually 15 years. Refunds based on connections: ¾-inch tap $40, 1-inch tap $70, 1½-inch tap $170, Fire hydrants $85.

Fort Wayne, Ind. No Standard Plan. Method may be: 1—Gross Return 18 per cent; 2—Gross Return 10 per cent in 10 years; or 3—Entire Cost for line outside City. In a period of 10 years, the department will refund from revenue received for a ¾-inch tap $50; 1-inch tap $100; 1½-inch tap $300; 2-inch tap $300; 3-inch tap $600; 4-inch tap $1200. 4—Old contract required developers to provide 75 per cent of funds. No refunds. Amount paid is considered as advance revenue and charged off into earnings about 20 percent per year until used up. 5—Consumer pays entire cost and gets a refund each time a lot becomes an active consumer for 5 or 10 year period.

San Bernadino, Cal. Financed by department without cost for five consumers per block; if less than five, deposit required. Deposit refunded when number of consumers reaches five.

Coffeyville, Kansas. Financed by surplus bonds; major additions financed by general obligation bonds.

Pawtucket, R. I. Bond Issue.

Haverhill, Mass. Owners of property along line guarantee 6 per cent.

New Orleans, La. Financed by Special 2 mill tax.

Canton, Ohio. Capital Expense Fund set up from Revenues (Sometimes $100,000/year).

Hartford, Connecticut. Assessment (payments may be made within 3 years). Assessment based on 5 year average cost of 8-inch pipe.

Tucson, Arizona. Major extensions financed out of general obligation bonds. Current extensions financed out of budget and special levy.

Little Rock. 1—No Deposit Necessary: Department will lay, at its own expense, 100 feet of 2-inch cast iron main for each new guaranteed consumer. Estimated Cost $50; Guaranteed Return $15. 2—50 per cent Refund Plan: Department expends approximately $50 per new consumer; balance or excess is paid by consumer; and consumer is refunded 50 per cent of revenue received from line (10 yrs.). 3—Consumer deposits cost of line in excess of 3½ times the immediate new revenue; additional refunds are made equivalent to 3½ times annual revenue for each new connection for 10 years.

REDUCING UNACCOUNTED-FOR WATER*

By Frank J. Broz, Superintendent
and
George Diamond, Engineer

(*Water Department, Cicero, Illinois*)

One of the essential factors in maintaining adequate service pressure is to preserve as nearly as possible the original tightness of a water distribution system. To accomplish this purpose the town of Cicero has organized a crew consisting of 2 engineers and 6 assistants who are specially trained in the technique of measuring and locating underground pipe leakage. District surveys are made regularly and the principal methods for locating leaks involve the use of the rod meter, 1-inch and 2-inch meters, aquaphone leak detectors, and a geophone. In addition to the underground leakage survey, a careful examination of all inside piping, plumbing and meters is being made by a master plumber employed by the department.

Underground leakage surveys, resulting in an average water saving of 3,000,000 gallons daily, have not only afforded an appreciable and immediate financial return, but also play an important part in helping to meet the financial program because of the recent water-works extension project completed with P.W.A. assistance. This program included 20 miles of new mains ranging in size from 8-inch to 30-inch, a 1,000,000 gallon elevated storage tank, the placing of 580 new hydrants, and the installation of an automatic pumping station of the latest design. The cost of the improvement ran upwards of $1,300,000, so that any saving in unaccounted for water tends to lessen the burden of the department. In addition, regular maintenance costs are reduced because every leakage survey includes an inspection of all valves, hydrants and service pipes on the mains tested and thus defects may be located and the necessary repairs made.

* Presented before the Wisconsin Section, November, 1937.

NATURE OF IMPROVEMENT

At this time I would like to touch briefly on the new improvements made to the water system to give a better idea of the importance of eliminating unaccounted for water in Cicero. With the financial assistance of the Public Works Administration and the supervision of Chas. DeLeuw and Co., Engineers, the following improvements were made:

A modern pumping station was constructed, and a 1,000,000 gallon elevated storage tank added, eliminating three pumping stations which were formerly used. Four high-efficiency, electrically-driven centrifugal pumps were installed with the following capacities: One pump, with a capacity of 10,000,000 gallons per day; two pumps, capable of pumping 5,000,000 gallons per day each; and, one pump capable of delivering 3,000,000 gallons per day. These pumps are controlled by a Westinghouse automatic switch board, which through the use of pressure switches, will automatically cut in or out of service the proper sized pump or pumps as the consumption increases or decreases, thereby maintaining a constant pressure of 45 pounds. Through careful planning, with regard to friction losses and differences in elevation, uniform pressures are maintained throughout the town.

The 20 miles of new main installed ranging in size from 8-inch to 30-inch is solely for the purpose of distributing water equally, and taps are not allowed on these mains with the exception of sprinkler system lines. This, in my opinion, is an ideal arrangement in many respects. In our underground leakage surveys we are able to measure through these lines and in that way eliminate about 45 percent of the ordinary operation, and a greater part of the inconvenience to the property owner.

The importance of eliminating unaccounted for water cannot be too heavily stressed. The success of the water department and the payment of the loan for the recent waterworks improvements are very dependent on keeping unaccounted for water at a minimum. Cicero received an outright grant of $425,000.00 from the United States Government and issued Water Revenue Bonds in the amount of $990,000.00 to pay for this improvement.

SURVEY METHODS

The town is divided into eight major districts. Each district is valved off so that one line may feed the entire district. On this

line is installed a set of taps. The first step is to measure by means of the rod meter, the entire district for a 24-hour period with all services open. A reading is taken every minute for the 24-hour period. A curve is drawn showing the maximum rate in gallons for 24 hours, the minimum night rate, and the total consumption. It is then possible to determine the importance of further investigation of the district. Taps are placed in convenient locations inside the district so that the major district can be subdivided into 2 to 4-block sections. A 1-inch tap is placed on either side of the valve and the water is passed around the valve through a 2-inch meter. All house consumption is eliminated by shutting off service lines at the curb stops so that the measurement includes only unmetered water. From one gauging point it is possible to measure a mile of pipe, depending of course on the size of the main. After all live services are shut off, valves are then closed at five minute intervals and the readings taken which will show the loss between each valved-off section. Rates may then be determined. This will indicate the method needed to locate and account for the rate. The first step in locating leaks is to listen with an aquaphone on valves and hydrants. Often times a noise on a hydrant will indicate not only the hydrant leaking, but a service or joint leak on the main. Next, by listening on the curb stops it is possible to tell which one is leaking or the possibility of a corporation leak or a leak on the main itself. Upon digging up the place where the leak is indicated it is possible that a leak will not be found even though a loud noise is noted on the service line. We then try the curb stops directly opposite this point and determine which is the loudest. Then the geophone is used on the surface of the pavement directly over the main and by process of elimination the leak is pinned down. The leaks after being located are dug up by W.P.A. labor and repairs are made by local licensed plumbers. Through experience we have found that 85 percent of our trouble occurs on service lines and 15 percent on the mains.

CAUSES FOR LEAKS AND REMEDY

Any number of causes for leaks may be given, some being typical of the section of the country in which they occur. One very common cause seems to be the failure of plumbers to allow the proper amount of slack in lead pipe at the corporation cock, which should form a 10-inch gooseneck. When the ground settles, the wiped joint, if too short, will not withstand the pressure of the earth and will

crack off. In the new town ordinance now being drafted, steps are being taken to eliminate this trouble. Our plan is to require the tap to be made horizontally with the main and a 2-inch cast iron pipe casing to extend from the main and cover the pipe 2 feet out. In this way the corporation cock and the wiped joint on the corporation coupling will be protected from earth settlement. Another common cause is the improper tinning of pipe in preparation for the wiped joint. This has now been entirely eliminated through rigid inspection.

Knowing the importance of keeping a water-tight system of mains and services, it is equally important to account and get *paid* for the water distributed to the consumers. This we have done to the extent of 82 percent of the total purchased from Chicago for period beginning January 1st, 1937 and ending October 31st, 1937. We take pride in the methods employed in accounting for such percentage, which are as follows:

First: Installation of the proper size and type of meter.

Second: Using the battery system of disc meters instead of compounds.

Third: Inspection and accuracy control of meters on monthly accounts or meters in sizes 1 inch and up.

Fourth: Periodical testing of $\frac{5}{8}$-inch to $1\frac{1}{2}$-inch sizes and annual testing of meters in sizes 2-inch and up.

Fifth: Repairing meters in the proper manner so as to insure accuracy on small rates of flow.

Sixth: Control of fire-line services by metering same.

In explanation of the above methods:

First: We determine the proper size and type by the use of a Recording Register, which gives the maximum and minimum gallon demand on such service.

Second: We obtain more revenue from the battery installation of disc meters than by using compound meters; also by doing so, our customer's service is never disturbed. He has service at all times. We eliminate working after hours, Saturday afternoon, etc., and the payment of overtime to our employees engaged in such work. We are now using a back pressure valve set at 2 pounds, installed on the outlet of one or more meters, as the case may be. This arrangement confines the small flows to one meter in the battery.

Third: Meters 2-inch and up are tested annually. We employ two men, specially trained, whose duty is to keep all meters

on monthly accounts as nearly as possible to the 100 percent mark. We have at the present time 545 meters on monthly accounts.

Fourth: Our latest method calls for a periodical test on $\frac{5}{8}$-inch to $1\frac{1}{2}$-inch sizes, inclusive, every five years, and in less time if the consumption reaches a point where the meter should be removed for inspection, repair and test. We are guided by a chart showing the maximum yearly registration allowable in all sized meters, also a chart showing the registration at which meters should be removed for inspection, repair and test.

Fifth: Our men in charge of meter repairs have had special training in their work, and no meter leaves the department until it has fully complied with the American Water Works Association Standard Specification tests, which I am sure you all know, are *too liberal.*

Sixth: The only way to control fire line services is by the use of meters approved by the Board of Fire Underwriters.

We now have in the course of construction, a meter shop, and when completed it will include testing equipment capable of testing up to the 6-inch size.

We will be able to cut down our Repair Department expenses with the installation of a test bench capable of testing eight $\frac{5}{8}$-inch or six 1-inch meters at one time. It is our earnest desire within a short time to be able to account for more than 82 percent of the water which we purchase from Chicago.

SUMMARY

The results of the work done in Cicero are:

A savings in purchases of water from the City of Chicago for period of 10 months (January 1, 1937 to October 31st, 1937), as compared to like period of 1936, in the amount of $31,936.40 or 62,620,400 cubic feet of water. Reduced to daily savings, it means that we are buying 171,563 cubic feet (1,286,722 gallons) less water each day.

Increase in sales of $20,598.34 during 10 month period over first 10 months of 1936.

In conclusion, for successful operation of water works, two things are necessary: 1st, eliminate leakage in your mains and services; 2nd, keep your meters as near 100 percent as possible. If we can do it, you can.

RELIEF FROM WATER HAMMER*

By Lewis H. Kessler

(*Assoc. Professor Hydraulic & Sanitary Eng.*)

(*University of Wisconsin, Madison*)

Water hammer is still a "bugbear" to water works men. It seems to be forever with us in causing breakage and leaks in the distribution system. It is of concern to the superintendent because his service of supply is closely allied with the work of architects, engineers, and plumbers in making satisfactory distribution systems in buildings. Stresses are set-up in the pipes, control devices and modern appurtenances connected to the piping system. These items, in general, are not designed for shock or water ram. Nuisances due to noises become apparent, often more so as the age of the installation increases or modern quick closing valves are installed. Fixtures and piping sometimes have to be taken out of service at inconvenient times. Decorations, plastering, and floors and stocks may be spoiled because of leakage. Opening of joints in pipe lines; slitting of house service laterals; increased stress on valve bonnets, packing glands and hydrant valves—all increase losses of water and cost of maintenance. Breaks due to water hammer occasionally have caused property damage by flooded basements and temporary loss of service of this portion of the structure.

It is therefore not unusual that we find reported in the technical press certain indications that the first work on water hammer took place in water works distribution systems. Eminent engineers and scientists have continued investigations on the phenomenon of water hammer whenever practicable. The investigations have been spurred on since the classic works of J. M. Michaud, 1878, Hugonoit, 1887, Joukovsky, 1898, Rateau, 1900, and Lorenzo Allievi, 1903. A symposium on water hammer arranged by the A.S.M.E. Committee on water hammer was held at Chicago in 1933 under joint auspices of the A.S.M.E. and the A.S.C.E. This symposium developed a most comprehensive summary of the reported theory and its

* Presented before the Wisconsin Section, November, 1937.

verification by experiments on the characteristics and behavior of water hammer. A study of this treatise will indicate the complex nature of the problem in general and the difficulty encountered by the water works superintendent in predicting and controlling water hammer. It also indicates the need for further research for aids in improving methods for relief from water hammer.

The phenomenon of water hammer is described as follows by Geo. M. Peek (Trans. A.S.M.E., Dec., 1899): "When a liquid is flowing in a pipe, there is a definite amount of energy in the liquid, and if we stop the flow, this energy must be used up in some way. If the liquid is incompressible, and if we stop the flow suddenly, the energy of the liquid is used up in doing work on the pipe by stretching it or increasing its diameter. If the liquid is compressible, the energy of the liquid is used up in compressing the liquid and stretching the pipe. Nearly all bursting of pipes is due to sudden checking of the velocity of the liquid or to the freezing of the liquid." The increase in pressure above the flow pressure in the pipe line while the water is flowing under steady conditions is called "water hammer." It is often, although not always, accompanied by a sound or clank like that of a hammer striking a pipe.

Theory involving considerable mathematics and several assumptions which may or may not be correct under some conditions, permit us to predict the maximum water hammer pressure likely to occur due to closure of a valve, faucet or control device in a layout that is not too complex. However, for practical purposes, hammer pressure of about 60 #/sq. in. above flow pressure can be expected to occur in small pipes for each foot per second of velocity of flow that is suddenly extinguished by closure of a valve. This means that an excess pressure of 600 #/sq. in. may occur, if a velocity of 10.0 ft./second is stopped instantly. Instant closure, in turn, is defined as that time required for the pressure wave to travel from valve to the source and return at approximately the speed of sound in water. The modulus of elasticity of the pipe, diameter, thickness, rigidity and fixed ends all modify the speed of the wave. In general, the advance of the wave front is from 4500 to 4000 ft./second in small steel and iron pipes. Closures of valves slower than instant closure also cause water hammer but of lesser magnitude. Valves which appear to close slowly actually can cause maximum hammer, because the effective closure takes place during the last 15 per cent or so of the valve movement.

Dead ends, according to some theories, may cause an additional effect on the pressure waves produced by water hammer, and result in hammer pressures higher than those in simple pipes. Theoretically, this increase can, under certain conditions, double the maximum water hammer pressure. So far, the writer has been unable to verify this in the laboratory under almost ideal conditions.

It has been proven in actual layouts within the laboratory that pressure waves can travel into the supply main and into adjacent house connections and buildings; these lines being in effect, dead ends. It is probable that this consideration is of more immediate importance to water works men than the dead ends in the large mains because home owners or tenants will enter complaints when water hammer is encountered in buildings, at least when the source is from outside the building. Water hammer originating in piping in buildings is likely to be much greater than that originating in street mains, due to the higher velocities carried under normal conditions.

RELIEF FROM WATER HAMMER PRESSURE

Having briefly discussed the occurrence and causes of water hammer, the writer will attempt to cover some of the means by which water hammer may be alleviated.

The first six items below enumerate the generally accepted methods of relief; the seventh is a new development, and the primary subject of this paper.

1. By closing all valves slowly particularly the last 15 per cent movement.
2. Spring operated relief valves.
3. Surge tanks.
4. Automatic surge suppressors actuated hydraulically or electrically.
5. Mechanical shock absorber or cushion.
6. Air chambers.
7. Mechanical-pneumatic arrester or fluid impact absorber.

All relief devices should be placed as close to the source of water hammer as is conveniently possible.

In recent years the water superintendent has realized that his responsibility relative to the distribution system does not end at the curb. Health hazards relative to cross-connections have shown him that he must be ever vigilant; concerned with both the successful

completion and with the proper and safe operation of his system into and including the dwellings, business and industrial establishments. Only in this way can he deliver a product safe for the health of mankind. His customers advise him when the plumbing system fails to produce adequate flow of water and he is the one expected to take action to remedy the situation. After years of contact with water works men in university short courses and elsewhere, the writer has learned that water hammer conditions that aggravate the home owner, and house engineers, also cause them to come to the superintendent for counsel and advice. Since he can neither control the way in which people operate the plumbing fixtures, nor generally specify the type installed, he is helpless to eliminate water hammer by having people close valves slowly (No. 1 Method of Relief). Besides, since the modern trend is towards quick closing faucets and automatic flush valves for both domestic and industrial use, and since certain industrial operations require these valves, it would appear that water hammer difficulties are on the increase. In some cases the superintendent can recommend relief valves where it is possible to bleed water from the system to an open fixture or drain. In others he can suggest the mechanical shock absorber which, due to its large inertia effect, has not proven entirely satisfactory. The fact is that he has usually come back to the old standby, the air chamber, even though he knows of the difficulty of keeping air in these chambers under repeated shocks. Unless provision is made for replenishing the air in chambers, this relief device has not met with continued success.

Little data, verified by experiment, have been available for the superintendent by which he could even recommend the proper size of an air chamber. Prof. R. C. Carpenter, of Cornell (Trans. A.S.M.E. 1893), gave some of the first data on air chambers and their behavior. Prof. N. Joukovsky at Moscow, 1897 (Translation by Miss O. Simin, Proc. A.W.W.A. 1904) did some important work on the effect of air chambers.

Dean M. L. Enger, University of Illinois, in the supplement to 1933 A.S.M.E. Symposium, using laws of mechanics of fluids and thermodynamics of air, developed his formula for required volume of air in an air chamber. He supports his conclusions by a number of experiments. W. S. L. Cleverdon, formerly Professor of Sanitary Eng., New York University, in his new book, *Plumbing Engineering*, (Pittman Publishing Co., New York, 1937, page 23), shows the

results of his tests and outlines his recommendations to the master plumbers concerning the use and installation of air chambers. It still remains for someone to corral available information and present it in its most useful form so that water hammer can be relieved by proper selection of an air chamber. When it functions, an air chamber is admitted to be the most economical and efficient protective device for the distribution systems of buildings.

For many years the staff at the University of Wisconsin Hydraulic and Sanitary Laboratory has been interested in finding a device that would accomplish the results of an air chamber and at the same time would not have its inherent disadvantage. It was believed that the public would accept such a product providing the cost and installation was not prohibitive.

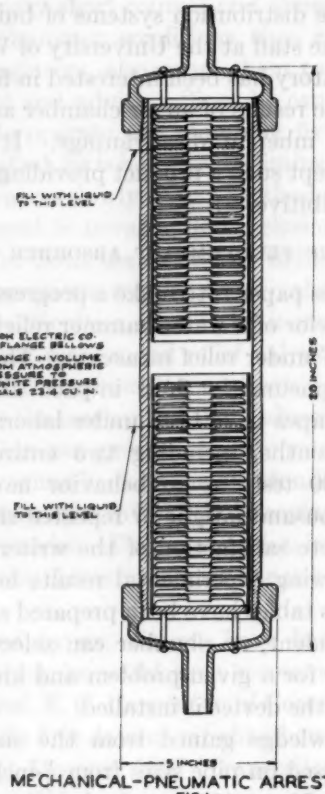
DESCRIPTION OF FLUID IMPACT ABSORBER OR ARRESTER

The purpose of this paper is to make a progress report on the characteristics and behavior of a water hammer relief device listed above as method number 7 under relief measures. This device is a combination mechanical-pneumatic fluid impact absorber or arrester. Several sizes and shapes have been under laboratory and field tests for the past 20 months, including two entire summer vacation periods. Over 4,000 tests as to behavior have been completed. Several hundred thousand fatigue or repeated shock tests have been made to the complete satisfaction of the writer. About 165 graph or curve sheets showing experimental results have been completed. From these analyses tables have been prepared so that the architect, engineer, superintendent, or plumber can select the proper size of absorber or arrester for a given problem and know what the results will be, if and when the device is installed.

Our present knowledge gained from the sizes tested, indicates that these can be used on pipe sizes from ½-inch to 2-inch diameter inclusive and from 50 to 400 feet in length. They have proven successful when used in multiple on pipe lines in hospitals up to 3-inch diameter handling water hammer caused by quick closing valves on laundry machines. The university contemplates a publication of these results. This paper will endeavor to indicate a small portion of the work accomplished and the application of results to relief from water hammer.

This unit, one type of which is shown in Figure 1, consists of an expansion chamber similar in shape and appearance to an air chamber,

and it can be connected in any manner to the hot or cold water pipe line as close to the source of water hammer as is possible. The expansion chamber contains one or more compression chambers constructed of fluted spring tempered bronze resilient discs so as to form a type of bellows construction particularly adapted to take water


FILL WITH LIQUID TO THIS LEVEL
COOK ELECTRIC CO. 50 FLANGE BELLOWS
CHANGE IN VOLUME FROM ATMOSPHERIC PRESSURE TO INFINITE PRESSURE EQUALS 23.4 CU. IN.
FILL WITH LIQUID TO THIS LEVEL
20 INCHES
5 INCHES

MECHANICAL-PNEUMATIC ARRESTER
FIG. 1

shock. The compression unit is partially filled with fluid and the remaining portion with air or gas under atmospheric pressure. The unit is properly sealed and supported by the expansion chamber. The complete unit will operate in a horizontal, vertical, or upside down position and it can be located parallel or at any angle to the pipe line and connected to it by usual pipe fittings.

HOW THE UNIT OPERATES

When the unit is on the pipe line, the expansion chamber becomes waterlogged in a manner similar to any air chamber after a period of use. The sealed compression unit containing fluid and gas is compressed as the water pressure is turned on to the system and the effective volume in the unit is decreased under this *static* pressure due to the water main pressure minus or plus the respective elevation of the arrester above or below the main. As the valve or faucet is opened and flow of water takes place, the compression unit expands according to the gas laws to *flow head* or pressure, and the effective volume of gas increases but never is equal to the volume at atmospheric pressure. As the valve is closed, the velocity in the pipe line is partially stopped, and increase in pressure or water hammer results. This valve closure compresses the layers of water nearest the valve and some of this water under compression is driven into the expansion chamber. The bellows compress according to the gas law and work is done by the water in compressing the gas in the unit.

As Dean Enger explains, in discussing air chamber behavior, the energy either stored or absorbed in the compression unit is (1) equal to the kinetic energy of the water in the pipe before the valve is closed, plus (2) the work done by the flow pressure after the valve is closed, minus (3) the energy required to compress the liquid and stretch the wall of the pipe, minus (4) the energy dissipated in pipe friction in the passages to the air chamber, and minus (5) any work done in lifting liquid into the expansion chamber depending on amount of water present and position of the arrester. Essentially, this explanation holds for the arresters under discussion.

It is possible for water to flow into the expansion chamber from the valve to the arrester connection and from the source of supply to the arrester connection. Since the compression wave due to valve closure no longer travels from valve to source undiminished in magnitude of pressure, the maximum water hammer pressure will be reduced depending upon the size of the unit or the amount of gas contained therein under flow pressure just before the valve was closed. Noise and vibration of the piping and stress on the piping and connected fixtures is materially reduced.

It is important to note that as the bellows start to move a small differential in pressure exists in the expansion chamber over that in

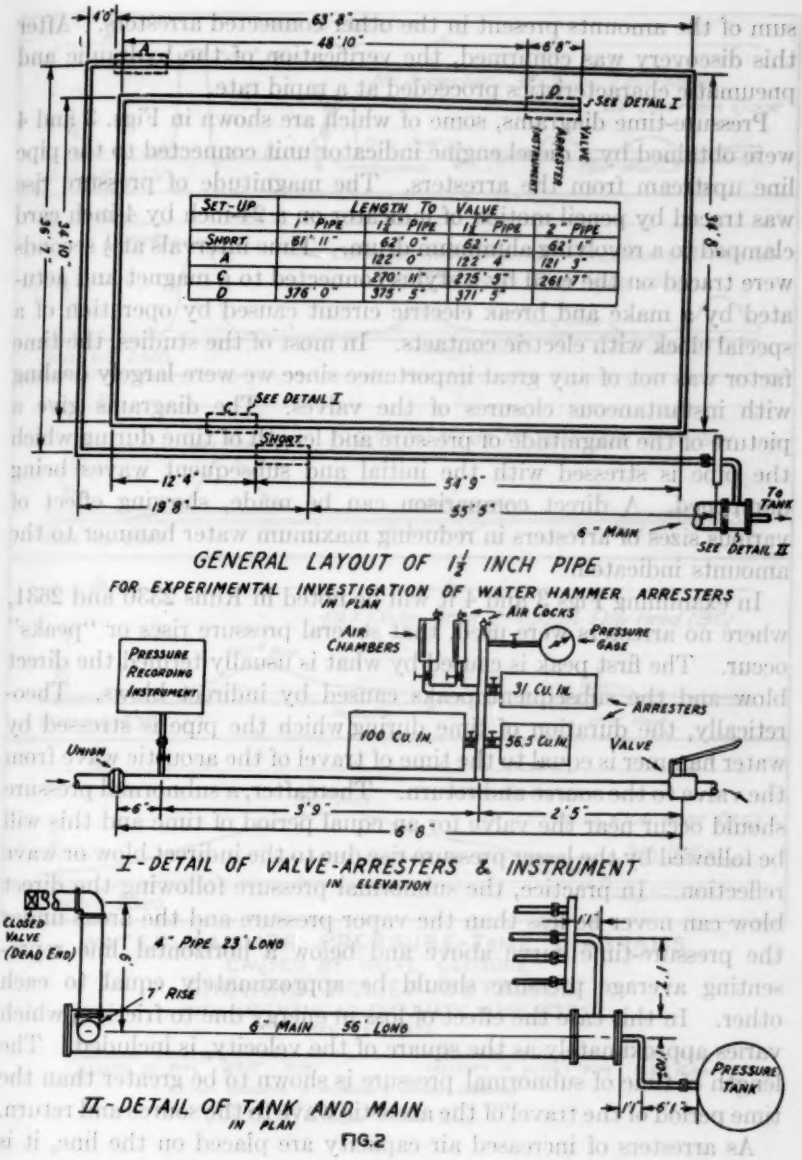
the compression unit, otherwise the bellows would not move. The bellows construction is designed to permit some differential in pressure. At point of maximum compression of the bellows, pressure is equalized both inside and outside of the bellows. Little stress is set up in the bellows or joints during movement and, assuming that the fluid within the bellows acts in a manner similar to a mechanical stop, the unit could, theoretically, take infinite pressure without failure. The movement of the various discs making up the flanges in the bellows can be likened to bending a sheet of paper, i.e., without appreciable stress.

Since pressures in the device are essentially balanced at all times, there is no force trying to make the gas escape. Any differential would tend to confine the gas inside the compression unit. These characteristics peculiar to the design and construction of the unit have been verified by the fatigue tests. The writer is convinced that these units as now designed can never be broken or distorted to the point of failure by water hammer pressure regardless of its magnitude.

TESTS CONDUCTED

Fig. 2 indicates one of the layouts used in testing. A calibrated pneumatic pressure tank supplied with air and water from the laboratory was connected to a 6-inch main comparable with ordinary water main service. This main supplied a header or manifold from which the various sizes and lengths of pipes were run. Five lengths of pipes from 50 to 400 feet were tested for ½-, ¾-, 1-, 1¼-, 1½-, and 2-inch pipes. The required static head to produce the several velocities from 3.0 to 20 feet per second was determined and manually controlled for each of the 4,000 test runs. The arresters or absorbers were all located at a fixed distance from the quick closing valves, quick closing faucets and flush valves used in producing water hammer in these tests. The water hammer pressure always remains a maximum value between the valve and the arrester for a time period of at least a double traverse of the acoustic wave which travels about 4,500 ft./sec. for the pipes under these tests.

Various sizes and numbers of arresters were used from 3.0 cubic inches, to 134 cubic inches volume of gas at atmospheric pressure. Several sizes of calibrated air chambers were also placed on the line at point of pressure-time measurement to extend the experimental data to limits even beyond practical interest. It was found that no



GENERAL LAYOUT OF 1½ INCH PIPE
FOR EXPERIMENTAL INVESTIGATION OF WATER HAMMER ARRESTERS
IN PLAN

I - DETAIL OF VALVE-ARRESTERS & INSTRUMENT
IN ELEVATION

II - DETAIL OF TANK AND MAIN
IN PLAN

FIG. 2

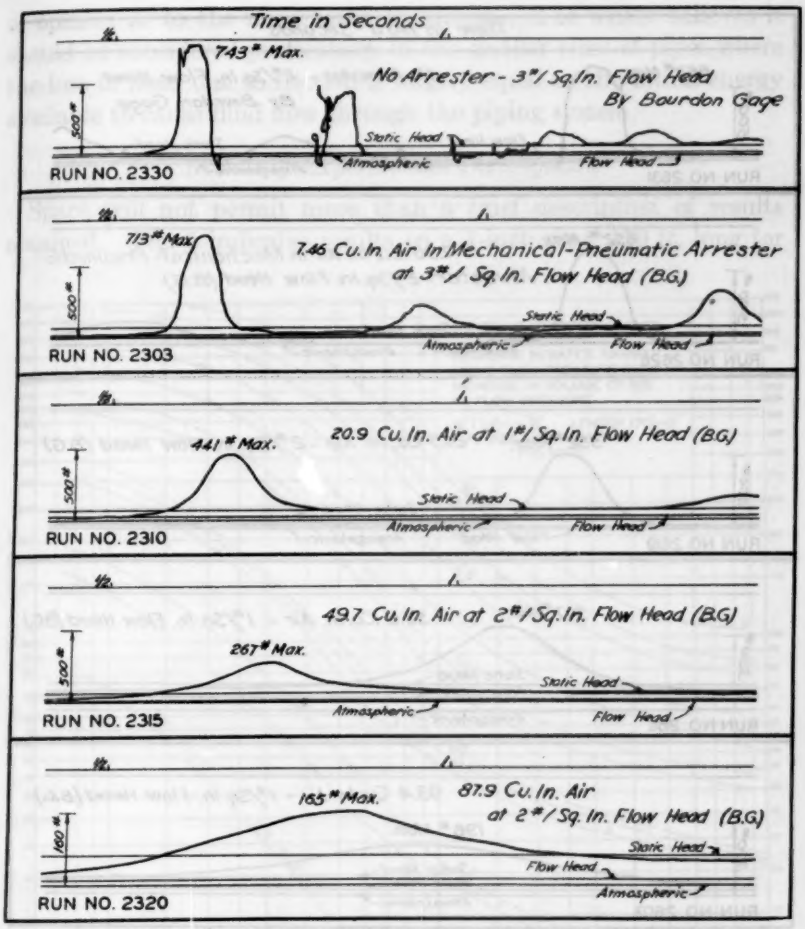
matter the position, as each arrester was placed into service by opening the gate valve connecting it with the pipe line, the respective amount of gas in the arrester at *Flow Pressure* could be added to the

sum of the amounts present in the other connected arresters. After this discovery was confirmed, the verification of the hydraulic and pneumatic characteristics proceeded at a rapid rate.

Pressure-time diagrams, some of which are shown in Figs. 3 and 4 were obtained by a diesel engine indicator unit connected to the pipe line upstream from the arresters. The magnitude of pressure rise was traced by pencil motion of indicator on a 24-inch by 4-inch card clamped to a revolving aluminum drum. Time intervals at ½ seconds were traced on the card by a stylus connected to a magnet and actuated by a make and break electric circuit caused by operation of a special clock with electric contacts. In most of the studies, the time factor was not of any great importance since we were largely dealing with instantaneous closures of the valves. The diagrams give a picture of the magnitude of pressure and length of time during which the pipe is stressed with the initial and subsequent waves being dampened. A direct comparison can be made, showing effect of various sizes of arresters in reducing maximum water hammer to the amounts indicated.

In examining Figs 3 and 4 it will be noted in Runs 2330 and 2631, where no arresters were used, that several pressure rises or "peaks" occur. The first peak is caused by what is usually termed the direct blow and the subsequent peaks caused by indirect blows. Theoretically, the duration of time during which the pipe is stressed by water hammer is equal to the time of travel of the acoustic wave from the valve to the source and return. Thereafter, a subnormal pressure should occur near the valve for an equal period of time and this will be followed by the lesser pressure rise due to the indirect blow or wave reflection. In practice, the subnormal pressure following the direct blow can never be less than the vapor pressure and the areas under the pressure-time curve above and below a horizontal line representing average pressure should be approximately equal to each other. In this case the effect of loss in energy due to friction, which varies approximately as the square of the velocity, is included. The length of time of subnormal pressure is shown to be greater than the time period of the travel of the acoustic wave to the source and return.

As arresters of increased air capacity are placed on the line, it is obvious that a decrease in the magnitude in water-hammer pressure occurs both under direct and indirect blows. However, the length of time that the pipe line is stressed by water hammer is increased. The pressure-time diagrams indicate that part of the dampening



WATER HAMMER PRESSURE-TIME DIAGRAMS
CAUSED BY VALVE CLOSURE
SHOWING RELIEF OBTAINED
WITH MECHANICAL-PNEUMATIC ARRESTERS

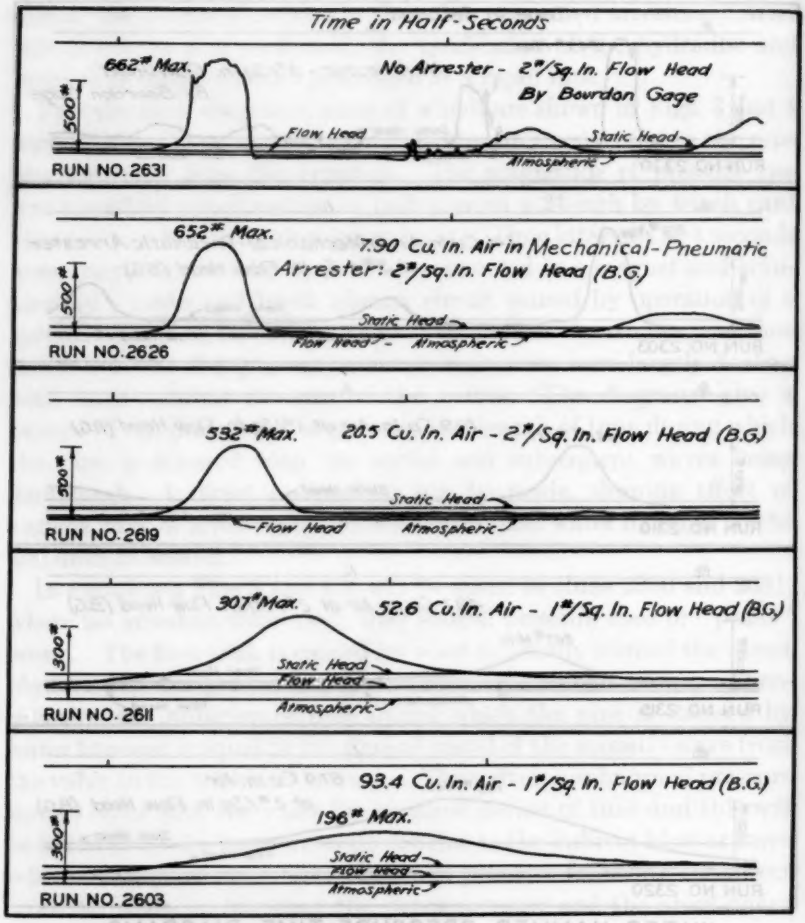
GENERAL DATA

NOMINAL DIAMETER OF PIPE 1¼"
LENGTH OF PIPE 123.67' - 11.2'
DISCHARGE AT FLUSH VALVE 3.1 G.P.M.
VELOCITY IN SUPPLY PIPE 11.95 ft./sec.

TESTS CONDUCTED AT HYDRAULIC & SANITARY LABORATORY
UNIVERSITY OF WISCONSIN JULY 1928
BY L.H. HEISLER G.A. RONLEN H. GARET C. SALICH

FIG. 3

effect of the direct and indirect blows can be attributed to friction. Since all analyses indicated in this paper are made from data taken by actual pressure-time diagrams, the effect of friction is included in the tabular data of Tables 1 and 2.



WATER HAMMER PRESSURE-TIME DIAGRAMS
CAUSED BY VALVE CLOSURE
SHOWING RELIEF OBTAINED
WITH MECHANICAL-PNEUMATIC ARRESTERS

GENERAL DATA

NOMINAL DIAMETER OF PIPE 1 IN. — DISCHARGE AT FLUSH VALVE 27.1 G.P.M.
LENGTH OF PIPE 223 FT. - 6 IN. — VELOCITY IN SUPPLY LINE 10.32 ft./sec.

TESTS CONDUCTED AT HYDRAULIC & SANITARY LABORATORY
UNIVERSITY OF WISCONSIN — JULY 1936
BY L.H. KESSLER G.A. ROHLICH H. GAMET C. SLICK

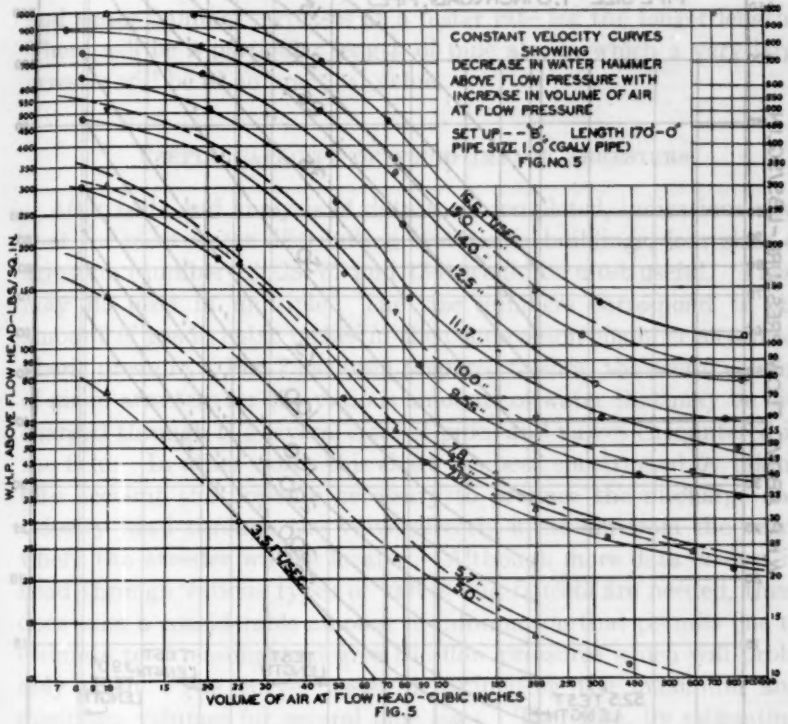
FIG. 4

Interlocking equations used to predict pressure-time surges that include the effect of friction become quite complex and in many computations, the effect of friction is neglected. There is a difference

of opinion as to the importance of friction. The writer believes it should be considered particularly in the smaller sizes of pipes where the loss in head due to friction is largely equal to the initial energy available to cause fluid flow through the piping system.

Results of Experimental Investigation

Space will not permit more than a brief description of results obtained. Fig. 5 indicates results on a 1-inch pipe, 170 ft. long for


CONSTANT VELOCITY CURVES SHOWING DECREASE IN WATER HAMMER ABOVE FLOW PRESSURE WITH INCREASE IN VOLUME OF AIR AT FLOW PRESSURE
SET UP - - 'B', LENGTH 170'-0"
PIPE SIZE 1.0" (GALV PIPE)
FIG. NO. 5
15.6 FT/SEC
15.0 "
14.0 "
12.5 "
11.17 "
10.0 "
9.45 "
7.91 "
5.7 "
4.0 "
2.5 FT/SEC
W.H.P. ABOVE FLOW HEAD - LBS/SQ. IN.
VOLUME OF AIR AT FLOW HEAD - CUBIC INCHES
FIG. 5

several velocities of flow. It shows the decrease in water hammer pressure above flow pressure as an increase in volume of air at flow pressure is used. The curves are "S" or "Ogee" shaped on logarithmic paper. A little gas in the compression chamber does very little good and the curves to the left approach maximum hammer. As a larger arrester is used, the pressure reduces almost in direct proportion to the amount of air available. Thereafter adding more air has very little effect because static head conditions are being ap-

proached. Enger's equation involving many terms will produce similar curves providing an experimental coefficient "m" in his equation has been determined for all sizes of pipe and various velocities.

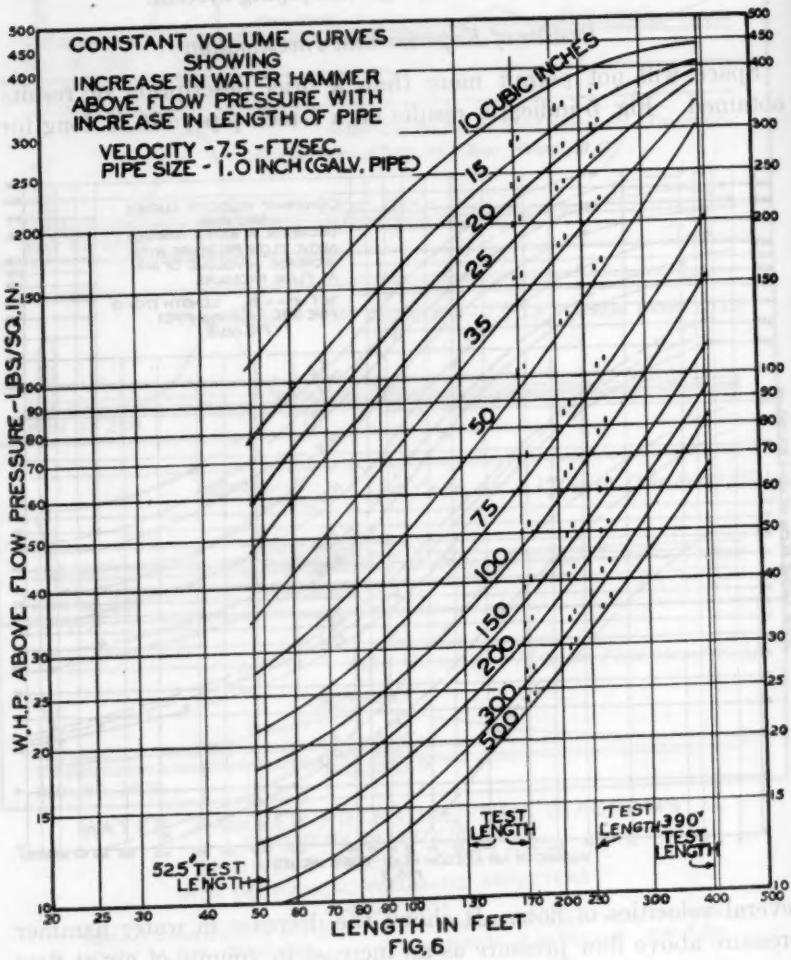


FIG. 6

No simple formula appeared probable so it became necessary to investigate effect of length of pipe, at the different velocities of flow.

Fig. 6 has been determined after "S" curves similar to Fig. 5 were obtained at 52.5, 130, 170, 230, and 390-foot lengths. Note that this figure applies only to a velocity of flow of 7.5 ft./sec. Curves at

other velocities have been constructed. For a constant volume of air, there is no simple relationship showing increase in water hammer pressure with length of pipe. At low volumes, the arrester becomes inadequate with increase in length of pipe because more energy is present in the moving water due to the increase in number of pounds flowing down the pipe. Maximum hammer is rapidly approached with increase in pipe length. There appears to be one constant volume at which pressure increase is proportional to length of pipe. Thereafter the larger volumes are more effective at reasonable lengths and water hammer increases at a faster rate for the longer lengths. There will be a particular length of pipe also in which a very large arrester will be of no practical value.

USEFUL CAPACITY OF FLUID IMPACT ARRESTERS

After tests and analysis of data were completed, indications were that for most water distribution systems in buildings, four sizes of arresters (number 13, 28, 67 and 134) would be most useful. These may be used in multiple. The size numbers correspond to the amount of gas in cubic inches in the compression chamber at atmospheric pressure. Fig. 7 has been prepared to show the useful volume of air in arresters for the various amounts of water that may be discharged through the several sizes of pipes and valves encountered in the field. In other words this chart has been constructed by taking into account the pressure necessary to produce the discharge and velocity head through the valve directly upstream from the valve where the arrester will be located. Although more data on loss in head through various types of valves and faucets are needed, there does exist a considerable amount of information that permits one to estimate to a reasonable degree the flow pressures which will probably occur. The curves assist in estimating the maximum and minimum volumes for several pipe sizes. Therefore, by estimating the discharge likely to be experienced, the useful volume of any of the four arresters or any combination of them can be obtained.

At this point it should be noted that the writer has tried to furnish a practical method by which the amount of relief may be determined for a given piping layout. It appeared that such a proposal should be made as non-technical as possible. He concluded that the use of tables and charts offered the best solution, since they could be interpreted without detailed knowledge of higher mathematics.

Indications are that the arresters referred to in this paper provide a more certain means of relief from water hammer than the air chamber. However, it should be understood that the use of this information

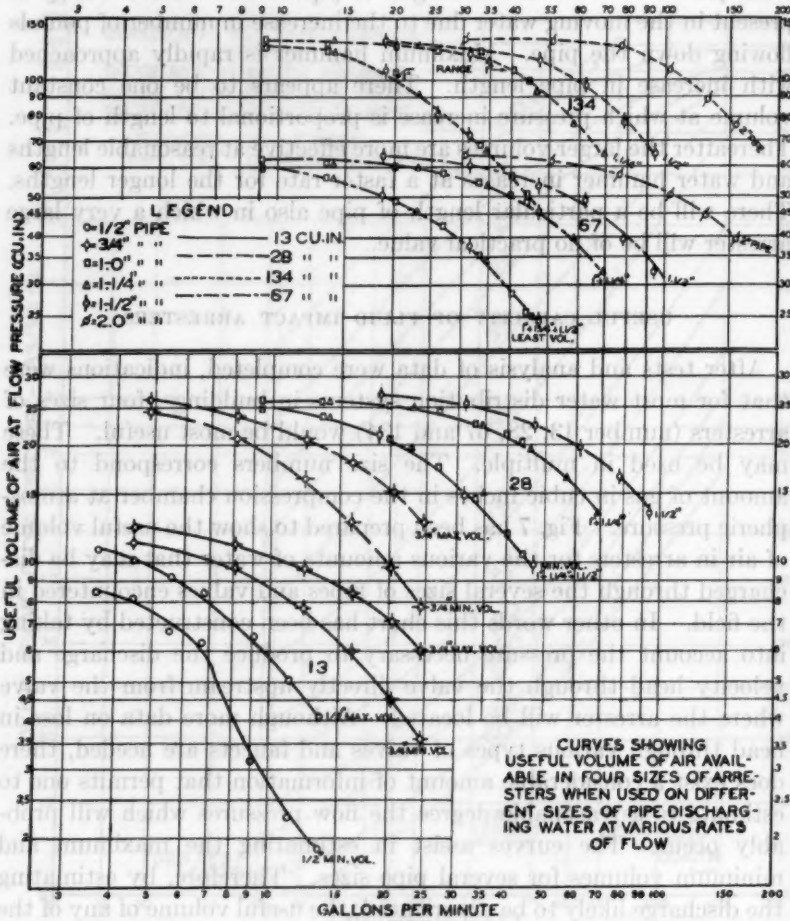

USEFUL VOLUME OF AIR AT FLOW PRESSURE (CU. IN.)
GALLONS PER MINUTE
LEGEND
13 CU. IN.
28 " "
134 " "
67 " "
134
67
28
13
CURVES SHOWING USEFUL VOLUME OF AIR AVAILABLE IN FOUR SIZES OF ARRESTERS WHEN USED ON DIFFERENT SIZES OF PIPE DISCHARGING WATER AT VARIOUS RATES OF FLOW

FIG. 7

is left to the individual and these data are equally applicable to air chambers, providing that the air chambers have a constant source of compressed air.

Following is a discussion of the use of some of the curves and charts:

EFFECT OF LENGTH ON SIZE OF ARRESTER REQUIRED

Tables 1 and 2 are two of thirty-eight that have been prepared to show the increase in magnitude of water hammer with increase in length; and decrease in water hammer with increase in useful volume of arrester at flow pressure. Note these tables apply to 1-inch pipe discharging 27.0 and 33.6 gallons per minute respectively. In table 2, regardless of length of pipe, the maximum hammer cannot exceed 762 pounds above flow pressure and the minimum static head to give

TABLE 1

1-inch pipe. 27 gallons per minute. 10 feet per second

Length in feet.....	50	75	100	125	150	175	200	300	400
Maximum water hammer pressure possible above flow pressure.....	← 610. →								
Min. static head.....	12	18	24	30	36	42	48	72	96
Volume of air required at flow pressure	All pressures in lbs. per sq. in. above flow pressure with arrester in use								
cu. in.									
10	240	340	415	470	510	540	565	610	610
15	175	265	340	395	440	480	505	575	600
20	135	210	275	330	380	420	450	540	590
25	110	170	225	275	320	360	400	505	575
35	79	125	170	210	255	290	330	450	550
50	57	87	120	150	180	210	245	365	480
75	40	56	74	93	115	135	160	265	380
100	32	44	56	70	86	102	120	195	280
150	26	33	41	50	60	72	84	145	220
200	23	28	35	42	50	59	69	115	175
300	20	25	30	36	43	50	58	94	135

the 33.6 gallons per minute has been shown, in order to assist in checking discharge assumptions or calculations. If this static head is not available, plus an amount for losses in the valve, the designer cannot possibly obtain the discharge shown under general data in the table. The values of pressures indicated for the several lengths are those that will occur under instantaneous valve closure when arresters are used having useful volumes shown in the left hand column. These tables can now be used in conjunction with Fig. 7 to select an arrester.

Let us assume 33 G/M discharge through a 1-inch pipe 125 ft. long to a flush valve and that we wish to reduce the pressure from 762 pounds to 580 pounds. Fig. 7 indicates Size No. 28 has a minimum volume of 15 cubic inches which should reduce the pressure to 580 pounds. If the flush valve has a low loss in head, useful volume may be about 23.5 cubic inches and pressure would be reduced to about 435 pounds. This will give the range of utility. Size No. 67 will have a useful volume between 35 cubic inches and 56 cubic inches and in this case pressures according to Table 2 would be reduced to

TABLE 2

1-inch pipe. 33.6 gallons per minute. 12.5 feet per second

Length in feet.....	50	75	100	125	150	175	200	300	400
Maximum water hammer pressure possible above flow pressure.....	←				762.				→
Min. static head.....	19	28.5	38	47.5	57	66.5	76.	114	152
Volume of air required at flow pressure	All pressures in lbs. per sq. in. above flow pressure with arrester in use								
cu. in.									
10	260	480	600	665	705	730	750	762	762
15	200	355	485	580	645	690	725	762	762
20	160	290	415	510	575	625	660	730	740
25	127	235	345	435	505	555	600	680	700
35	90	190	280	355	415	470	510	620	670
50	64	125	195	260	320	375	425	545	590
75	49	86	127	170	210	250	285	410	495
100	40	67	96	127	157	187	215	325	415
150	34	54	75	96	118	140	160	240	310
200	29	43	58	75	92	110	127	195	250
300	25	36	47	59	71	85	98	150	200

from 355 pounds to less than 260 pounds. If size No. 134 is used at 70 cubic inches, pressure would be reduced to 127 pounds.

EFFECT OF LENGTH WHEN COMPOUND PIPES ARE USED

A number of tests were conducted with arresters located on compound pipe lines i.e. single pipe line composed of several diameters of pipes in series. The valve in each case was located at the end of the pipe of smallest diameter. The total pounds of water flowing is the same in each section under steady flow, but the kinetic energy in the

water flowing in the large pipe is less than what it would be if the pipe were of smaller diameter. Hence, less energy must be absorbed or dissipated.

Our tests, by no means complete, indicate for the present until further information is available in small pipe lines, the following rule of thumb can be followed. If 30 percent of the length of the larger pipe is added to the length of the smaller pipe, the "equivalent" total length can be used to enter the tables for arrester selection. The short pipe stub from riser pipe to fixture (usually of smaller diameter than riser pipe) should not be considered as causing compound pipe behavior.

USE OF TABLES TO SELECT SIZE OF RELIEF DEVICE

Table 3 is one of several that have been prepared for those who find reading and interpreting of tables easier than charts. Velocities common to all these sizes of pipes are indicated at the top of each column and interpolation between values is close enough for practical purposes. Gallons per minute is indicated for the respective velocity in each size of pipe. The range in flow pressures likely to occur is shown as well as the accompanying useful volumes in the arrester at these pressures.

Using the example as before, assuming a 1-inch pipe, 125 feet long, discharging 33.6 gallons per minute to a flush valve, we find from table 3 that useful volume of size 67 is 36.0 cubic inches. Table 2 shows that 35 or say 36 cubic inches of gas will reduce water hammer pressure from 762 #/sq.in. to 355 #/sq.in. Other types of valves in general will have a lesser flow pressure than the flush type valve, and table 3 shows that the useful volume could be as high as 56 cubic inches. In this case the water hammer pressure from table 2 would be reduced to 260 #/sq.in. or less.

Just to give a rough idea as to the range of utility of several sizes of arresters, Table 4 is shown. It applies to various lengths and sizes of pipes, but only for a velocity of flow of 10 ft./sec. and, providing the user can accept pressure reduction from 600 #/sq.in. down to 200 #/sq.in. or two-thirds reduction. We find from this table that the limit of utility of each of the four arresters to bring about a pressure reduction down to 200 #/sq.in. is indicated by the size that is italicized. Note that size 67 means 67 cubic inches of air is available at atmospheric pressure in either the arrester type of relief device or in air chambers.

TABLE 3

For size No. 67 water hammer arrester

To determine *Useful Volumes* or *Capacity* of Arrester When Installed on 1″, 1¼″, 1½″ and 2″ Pipes discharging water through valves or faucets to fixtures at various rates. (Flow Pressure = Pressure directly upstream from valve when water flows under steady conditions. Values given to be used only when actual data is not available.)

Velocity (ft./sec.)	3.0	3.5	5.0	7.5	10.0	12.5	15.0	16.5
1-Inch Pipe								
Gal./Min		9.4	13.5	20.2	27.0	33.6	40.4	44.4
High Flow Press. #*		1.0	1.9	4.4	8.2	12.8	19.0	23.0
Low Flow Pressure #		0.5	0.7	1.0	1.6	2.5	3.8	4.6
Cubic Inches								
Low Useful Volume*		*62.0*	*58.0*	*50.0*	*43.0*	*36.0*	*29.0*	*26.0*
High Useful Volume		64.0	63.0	62.0	59.0	56.0	50.0	47.0
1¼-Inch Pipe								
Gal./Min	14.0		23.3	35.0	46.7	58.4	70.0	
High Flow Press. #*	2.1		6.0	14.0	25.8			
Low Flow Press. #	0.7		1.2	2.8	5.7	9.8	15.0	
Cubic Inches								
Low Useful Volume*	*57.5*		*47.0*	*35.0*	*25.0*			
High Useful Volume	63.0		61.0	55.0	48.0	*40.0*	*33.0*	
1½-Inch Pipe								
Gal./Min	19.0		31.6	47.5	62.3	79.2	95.0	
High Flow Press. #*	3.9		11.3	26.5				
Low Flow Press. #	0.3		1.0	2.8	5.4	9.5	15.0	
Cubic Inches								
Low Useful Volume*	*52.0*		*38.0*	*24.0*				
High Useful Volume	65.0		62.0	55.0	*50.0*	*40.0*	*33.0*	
2-Inch Pipe								
Gal./Min	30.8		51.4	77.0	103.0	129.0	154.0	
Flow Pressure #	0.3		0.8	1.9	3.8	6.5	10.0	
Cubic Inches								
Useful Volume	*65.0*		*63.0*	*58.0*	*52.0*	*46.0*	*40.0*	

* Flush or Flushometer Valve Requirements.

TABLE 4

For use as a guide in selection of size of water hammer arrester to reduce water hammer pressure from 600 % to 200 % or less

Velocity of 10.0 ft./sec. assumed in pipe lines of various lengths and diameters.*

Gallons per minute.....	9	16	27	47	62	103
Diameter of pipe.....	1"	1"	1"	1½"	1½"	2"
Length of supply piping in feet						
feet						
50	#13	#13	#28	#28	*#28*	#67
75	#13	#13	#28	*#28*	#67	*#67*
100	#13	*#13*	*#28*	#67	*#67*	#134
125	*#13*	#28	#67	*#67*	#134	#134
150	#28	#28	#67	#134	#134	*#134*
175	#28	#28	*#67*	#134	*#134*	
200	#28	*#28*	#134	*#134*		
300	*#28*	#67	#134			
350	#67	#67	*#134*			
400	#67	*#67*				

* This table is to illustrate in a general way the limitation of the range in usefulness of the four sizes of arresters proposed. Unless specific hydraulic data on the particular layout are available, arresters should not be recommended beyond the range shown in this table.

The 10′/sec. velocity has been taken because it represents conditions that will likely occur in practice where trouble with water hammer is a frequent occurrence. Other tables are available to show application of arresters to both higher and lower velocities when stopped by valve closure.

SUMMARY

The research project at the University of Wisconsin on characteristics and behavior of fluid impact absorbers or arresters has demonstrated that a new mechanical-pneumatic device can be made which is reasonably simple in construction and operation. It embodies all of the advantages of the air chamber for use in reducing water hammer pressure in the distribution systems of buildings. It is believed that the life of the unit will be equal to that of the life of bronze which has proven successful in the waters collected and distributed in American water works practice.

The compression unit or bellows, which is the most important element in the device, will always contain a definite and known

volume of gas or air available for service in protecting piping systems and the fixtures connected thereto.

Each unit has limitations as to its adequacy in relieving water hammer pressure depending upon the size or volume of gas contained in the compression chamber. Sufficient verification as to the behavior on pipe sizes from ½ to 2-inches inclusive and on lengths from 50 to 400 feet has been made by the 4000 tests conducted. These tests have made it possible for tables and charts to be constructed whereby the proper size of arrester can be selected quickly, for the given hydraulic conditions in the building. These tables can also be used to design the proper size of ordinary air chambers.

The absorbers can be placed in any position or connected in any usual way by the plumber, but they should be placed as close as possible to the valve causing water hammer.

When water hammer or shock originating in the water main distribution system is encountered or found to cause shock in the piping system of a building, the fluid impact absorber can be located at the entry of the house service into the building. This has been an old practice by superintendents in using air chambers. If the pressure is known or can be found, the tables can be used to indicate the size needed to reduce the magnitude in pressure to a given amount. For ordinary service it is believed one of the larger sizes of arresters will be adequate for relief from this water hammer coming from the outside source.

CONCLUSIONS

1. The water hammer pressure increases directly with the velocity extinguished by quick closures of valves or faucets.

2. The increase in pressure or excess pressure due to water hammer varies approximately in proportion to the square of the velocity of flow in the pipe when impact absorbers are used.

3. When air chambers or impact absorbers are used, the water hammer pressure increases approximately as the length increases. The law of behavior is difficult to determine and is not simple to use in making calculations.

4. Maximum water hammer pressure increases slightly with decrease in diameter of pipe because the smaller the pipe the larger the velocity of propagation of the pressure wave front.

5. Water hammer pressures increase approximately as the square of the diameter of the pipe when impact absorbers are used.

6. Laboratory tests indicate that the maximum water hammer pressure occurs just upstream from the valve and decreases slightly as the distance to the source increases.

7. Maximum water hammer pressures exist in the pipe line between the valve and the arresters regardless of the amount of relief provided.

8. Enger's formulae for useful volume required in air chambers are the best that have so far been reported to determine the relationship between the volume of air required and the water hammer pressure when velocity of flow, size and length of pipe, static head, and flow head are known.

9. Two or more arresters can be used to protect a given pipe line. Problems will arise in which a greater useful volume of air is required than can be furnished by just one arrester. In this case any number of arresters can be placed on the pipe line or on the same "tee" that is used for one arrester connection. The useful volumes of air in each arrester, under Flow Pressure, can be added together because this total amount is available for absorption of shock.

These conclusions are from thesis submitted for Masters' Degree in Civil Engineering, Univ. of Wisconsin Library, Madison, Wis. by Merrill B. Gamet and Honor Thesis by P. S. Davy, C. L. Miller and R. F. Zwettler.

The writer wishes to acknowledge the valuable assistance of Philip S. Davy, Research Assistant, in preparing this paper.

CATHODIC PROTECTION OF THE MOKELUMNE AQUEDUCT*

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An elementary knowledge of the nature and causes of corrosion is necessary for an understanding of the operation of cathodic protection. The corrosion, or loss of metal of buried pipe lines and similar structures in the soil is the result of a flow of electric current from the area being corroded. Stated in other words, underground corrosion of metallic structures is due to electrolysis. The origin of the current may be either external or within the soil itself. Stray current from electric railways is the chief offender in the case of externally produced electricity. It is now generally agreed that the only practical way to protect structures from stray current electrolysis is to control the current and prevent it from passing directly from the metal surface to the surrounding soil. Cross ties and drains at critical points are the means of accomplishing this control and pipe failures from electrolysis are practically unknown in a well bonded and drained system.

Many pipe lines are subject to electrolysis in locations remote from any external current source. In these instances the current is produced in the soil by galvanic action. Soluble minerals in the soil and water constitute the electrolyte. Water is a necessary component and conversely when water is present in the soil, corrosion in some degree, invariably takes place.

Galvanic soil currents usually originate from one of two causes, namely, dissimilar metals or dissimilar soils. In the first case the current, produced by a difference in the metallic structure, is localized in small areas. The resultant corrosion is referred to as local or self corrosion. The second type of current frequently travels a considerable distance, is known as "long line current" and is due to differences

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in soil conditions either because of moisture content or soil structure or both. An example of a condition favorable to this type of current production is a pipe line passing over hills and through irrigated fields or marshes. Current will flow on the pipe from the dry areas to the wet ones where it will leave the pipe with a consequent loss of metal, and return through the soil to the pipe at the top of the hill. Pipes traversing two different soils such as moist clay and sandy loam will lose metal in the clay.

Corrosion caused by these currents can be prevented, by "draining" the current from the pipe and returning it to the soil. Iron, steel, coke or zinc buried adjacent to the pipe and connected at points of normal current discharge by insulated conductors may be used for this purpose. The pipe sections to be protected by each station range from several hundred feet and upward in length and the current to be drained must be concentrated, usually to one connection. By selecting a metal for the rods that is anodic to the metal of the pipe, such as zinc, in the case of a steel or iron pipe, galvanic action between the ground rod and the pipe will "boost" the flow of current from the pipe along the connecting cable. The zinc beds will be consumed and will require frequent replacement.

If ground rods or beds of steel, iron or carbon are used, the flow of current from the pipe along the electrical conductor can be "boosted" by introducing a source of direct electric current in the conductor to concentrate the flow of current from the pipe to the soil through the grounding system (Figure 1). If sufficient current is drained from the pipe the potential or electrical pressure on the pipe will become lower than in the surrounding soil. With the electrical gradient sloping from earth to pipe, current will flow to the pipe from the ground. This current flow will neutralize or reverse current flow from pipe to soil at points other than the drain and prevent self-corrosion as well as that caused by "long line currents." This, in brief, is the action of cathodic protection, so named because the protected structure is considered the cathode of an electrolytic cell and the ground rod the anode. This form of protection is not protection in the same manner as that provided by a covering or coating, but is an actual control of the forces of corrosion. Perhaps a better term would be corrosion control.

Although the extensive application of this method of pipe protection is fairly recent, the cathodic protection of metals in an electrolyte is not new. The first application of electrolysis control

was the suspension of zinc plates inside steam boilers (1). The earliest reference to an application of this kind to come to our attention describes an installation made in 1891, but doubtless earlier experiments had been made. About 1910 or 1911 an Australian inventor patented an electrolytic process for the protection of metals (4). During the period 1911 to 1913 work was carried on by the Bureau of Mines (3) and in Australia, using an external source of electric energy. Some time after the year 1920 operators of gas

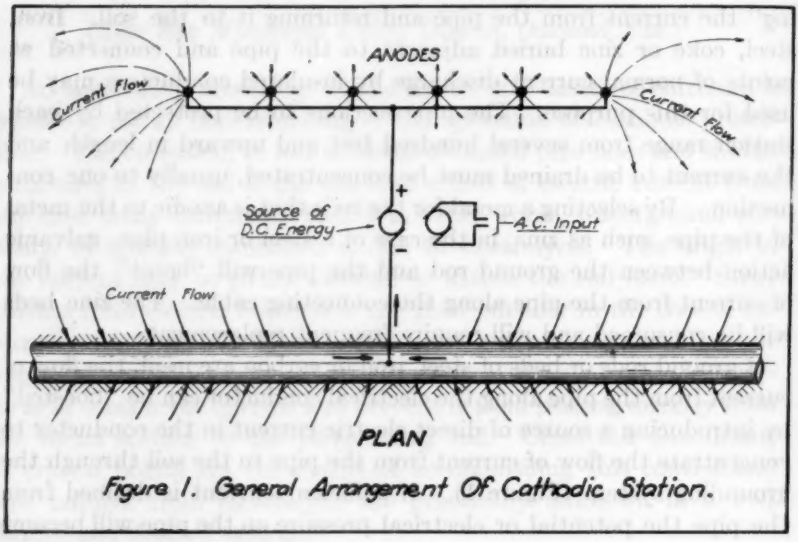


Figure 1. General Arrangement Of Cathodic Station.

and oil lines began to apply cathodic protection to their transmission mains.

THE MOKELUMNE AQUEDUCT

The East Bay Municipal Utility District's aqueduct from the Pardee Reservoir on the Mokelumne River to San Pablo Creek, was completed in 1928. The total length of this aqueduct, including tunnels, is 93.85 miles, of which 81.16 miles is steel pipe. This pipe is buried with the exception of $10\frac{1}{4}$ miles elevated on pile bents. The buried portion, 70.92 miles in length, consists of 70.36 miles of pipe 61, 63 and 65-inches in diameter in the ground and 0.56 miles of twin 54-inch pipe in 3 river crossings laid under the branches of the San Joaquin River. Soil conditions vary widely from dry rocky

formations in the foothills through sands and adobes to extremely wet peat lands in proximity to the rivers.

The buried portion of the pipe, outside the river crossings was installed with a protective bituminous dip coat and spirally wrapped felt covering. The pipe in the river crossings has a concrete jacket.

When the aqueduct had been in operation for about 3 years it appeared desirable to investigate the condition of the protective covering. Because of the length of the line, size of pipe and depth of earth cover, the cost of a comprehensive and satisfactory inspection by excavating would have been prohibitive.

TEST OF COATING

A method for testing the condition of the covering, without uncovering the pipe, was devised which proved to be entirely satisfactory and relatively inexpensive. It was assumed that the electrical resistance of the coating would vary directly with its condition, that is, coating in good condition would have a high resistance and broken coating a low resistance.

The equipment needed for such a test was simple and consisted of 36 cells of storage battery, a D.C. voltmeter with a 50 or 75 volt scale, a D.C. ammeter with a 2 ampere scale, a small variable rheostat for controlling the voltage, about 3000 feet of number 16 rubber covered copper wire on a reel with a crank, two $\frac{3}{4}$ inch x 4-foot steel rods, pointed at one end, two light sledge hammers, and a bar or jack to pull the bars out of the ground. With this outfit on a light truck three men were able to test about one and a quarter miles a day, taking a reading every 100 feet. The length of the test wire was determined by the average spacing of air valves and blowoffs which were used as a point of contact with the pipe.

The wire on the reel, the storage battery, rheostat, ammeter and rod were connected in series, with the voltmeter across the battery and rheostat as shown in Figure 2. The rheostat was adjusted from time to time to keep the voltage constant. The free end of the long wire was connected to the pipe in a manhole. One man drove the truck, read the meters, and recorded the data. The helpers drove and pulled the pointed rods every 100 feet and made contact on the rod while current readings were being taken. The voltage was adjusted to 50 volts at each test. A piece of light rope was used as a means of spacing the test rods. It is not necessary to drive the rods over the pipe. An inspection road is maintained along the Moke-

lumne Aqueduct, in the center of the right of way directly above the pipe. It was easier to drive the rods to one side of the road which at the same time eliminated the possibility of driving the rod to the pipe and puncturing the coating. This would have produced erroneous results. The test was conducted in the spring of the year, as soon as the surface of the ground had dried sufficiently to make going easy. The moisture content of the soil was still uniformly distributed so that results, independent of variable earth moisture, were comparable.

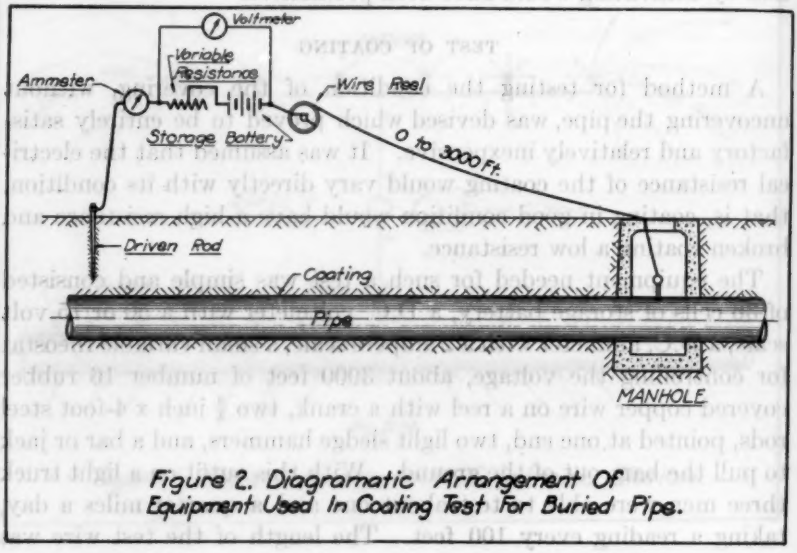


Figure 2. Diagramatic Arrangement Of Equipment Used In Coating Test For Buried Pipe.

It was not necessary to reduce the readings taken to ohms because all tests had been taken at the same voltage. The current in amperes varies inversely as the resistance in ohms, a high current value indicates a poorer condition of covering than does a low current. Values of current were plotted and locations of high readings noted.

Following the test, "sample" locations were excavated and the coating examined. The conditions found by direct inspection agreed with predictions made from the data. The wrapping had failed in tension along the top of the pipe due to stressing by the soil. The pictures in Figure 3 show the manner of failure. It was found that the failure due to soil stresses was generally the worst in the low places which are subject to the greatest seasonal moisture variations.

The extreme fluctuation of moisture content is responsible for movement of soil in contact with the pipe. With the exception of the river delta where the pipe is wet the year around, these locations are the most corrosive. In the delta, due to the continual submersion, water had penetrated the covering at the field joints and corrosion of the rivets had commenced in earnest. In the higher ground, where drainage is natural, the covering and pipe were in good shape.

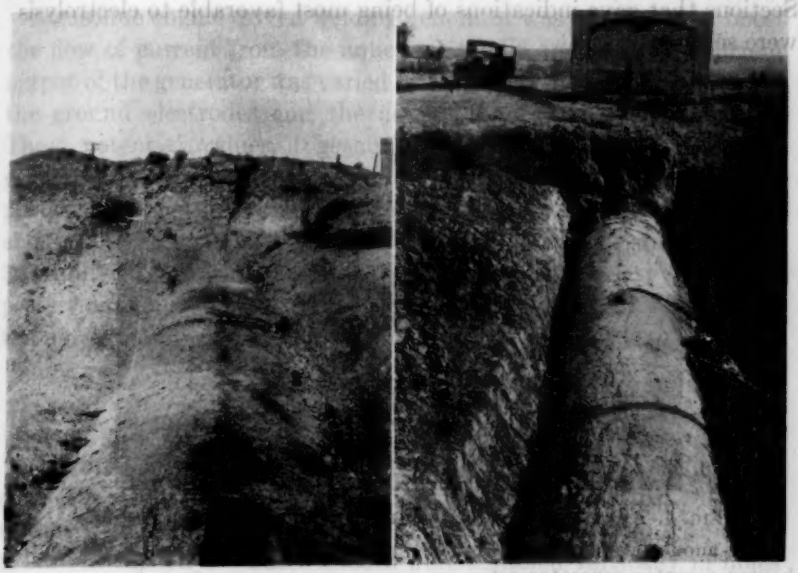


FIG. 3. SAMPLE EXCAVATIONS SHOWING CRACKS IN PIPE COVERING CAUSED BY SOIL STRESSES

The angle clip in foreground of left hand picture and cable in center of right hand picture are for attachment of coupons.

Corrosion had started along the cracks in the covering, at the field joints in the delta and in some cases on the lower side of the pipe where slings had bruised but not broken the covering during construction.

The corrosion had not progressed to a point where the pipe was in any immediate danger of failure, although some leaks due to corrosion had developed. It was evident that something had to be done, however, before conditions became serious.

At that time cathodic protection of oil and gas (8 and 9) trans-

mission mains had been operating a sufficient length of time to demonstrate its possibilities as a means of protecting pipe lines.

In the case of the Mokelumne Aqueduct it was out of reason to even consider excavating around the pipe and reconditioning or replacing the covering. Cathodic protection was adopted as the only practical means and thus far has proven entirely satisfactory.

The information obtained by the test and inspections provided a basis for determination of tentative locations for protection stations. Sections that gave indications of being most favorable to electrolysis were selected.

FIG. 4. ARRANGEMENT OF RECTIFYING EQUIPMENT AND ANODES

The anodes are marked with white posts. The cabinet in the left center houses a motor-generator set.

To obtain data from which booster current requirements could be determined further tests had to be made. In preparation for these tests ground electrodes, consisting of sections of well rusted $\frac{3}{4}$-inch iron pipe about 6 feet long, were driven into the ground along the edge of the right of way opposite manholes where the aqueduct could be easily contacted. Old, rusty pipe was selected, and further rusting promoted by watering after placing, to obtain a test point as near the ground potential and as free from polarization effects as possible. The iron oxide surrounding the metal and in close contact with the soil tends to eliminate electrolysis between the iron and earth. These ground electrodes were allowed to "age" after driving

for about a month when the potential differences between them and the aqueduct were measured with a millivolt meter.

In the meantime, anodes in the form of 2½ inch extra heavy pipe were installed at the proposed station locations. The material used was some second-hand pipe in stock, the size having no special significance. These anodes, each 20 feet long were set in holes drilled with a post hole auger. Usually two or three anodes were sufficient for testing each location.

A gasoline engine driven welding generator was connected to boost the flow of current from the aqueduct to the anodes. The current output of the generator was varied by steps and the potential between the ground electrodes and the aqueduct measured for each step. These potential values, together with the one taken before the generator was operated were plotted on a profile. This graph supplied the needed information for determination of booster capacity and zone of influence of each station. A potential difference of 300 millivolts of soil above pipe was considered necessary for proper protection. Booster voltage requirements are controlled largely by the anode area in contact with the soil.

EQUIPMENT

The first permanent anodes were the same as the preliminary ones—2½ inch extra heavy steel pipe in lengths of 20 feet. It was assumed that one anode would be required for each 4 to 5 amperes of station capacity. These anodes were installed vertically in holes drilled with a 4-inch post hole auger arranged in a line parallel to the aqueduct and spaced 25 feet apart. The vertical type was selected in order to have a part of the material in contact with wet soil throughout the year. In the season of the year when the water content of the soil is greatest the corrosion rate is a maximum, and naturally maximum protection is needed. Conversely, when the ground is driest the corrosion rate is least, and less protection is necessary. The vertical anode is ideal for such conditions. During the wet portion of the seasonal soil moisture cycle the entire length of the anode is in wet or moist soil, and the contact resistance is at a minimum. As the soil dries from the top down, the length of anode in wet soil progressively decreases, accompanied by an increase of contact resistance. With the impressed booster voltage remaining

about constant the current drained from the pipe to the soil will vary with the soil moisture.

After about six months of operation, some of the steel anodes had been consumed. That is, the metal below the low moisture line had been removed by electrolysis and gone into solution. Following this coke was used as material for anodes.

Holes 8 inches in diameter and in depths from 8 feet to 20 feet, depending on soil water conditions, were drilled, and filled and packed with coke and a 2 inch carbon rod, 80 inches long with an insulated cable attachment inserted in the upper end. Coke and carbon are relatively stable, electrolytically, and should serve as anodes for long periods of time.

After about two years operation the carbon rods are showing definite tendencies of failure, due to a breaking down of the bonding material. Experiments are being conducted with other types of binders and coke but sufficient progress has not been made to justify any conclusions.

The source of electric energy for each station is the distribution system of the public service corporation serving in the area. The energy is received as 60-cycle alternating current, single phase, and must be rectified to supply the direct current necessary for boosting.

A number of types of rectifying equipment are available, four of which are being used.

The larger stations, four in number, have an output capacity of 50 amperes. Motor-generator sets were chosen for these. Each unit consists of a two horse power motor driving a 1 kw. generator at 1800 r.p.m. The generator is designed for maximum of 50 amperes at 20 volts. Because the voltage required had not been definitely determined the first unit purchased was equipped with a 100 watt exciter to permit voltage variations. It was found that the exciter is not necessary, and the three later units were not so equipped. The advantages of a motor-generator rectifier are its stability, flexibility of voltage adjustment, freedom from temperature variation influences, ability to withstand some overload, and that is the one type of equipment for this purpose that has been proven by a long period of use. The disadvantages are brush, commutator and collector ring wear that must be taken care of for good operation.

One copper oxide rectifier, rated at 10 volts and 43 amperes, was installed. This equipment is not suitable for the continuous duty imposed by cathodic protection service. It has very little overload

capacity and is subject to failure from overheating. Although stations are not operated at overloads, being unattended, overloads from accidental causes, may arise and continue until the next inspection.

A number of vacuum tube rectifiers have been installed. The operative reliability of these units has been entirely satisfactory. Tubes have an operating life of from nine months to a year. These units are not as efficient as others in use.

One selenium type rectifier has been in operation for over a year. The operating characteristics of this unit are excellent for cathodic protection. No service of any kind has been required, and the efficiency is on a par with motor-generator sets.

CHECKING UP ON CATHODIC PROTECTION

It is quite essential to "check up" on the cathodic protection to be sure that corrosion is actually under control. The District maintains constant observation by daily inspection of each rectifier station, by periodic potential measurements between pipe and earth and by direct determination of corrosion rates.

Patrolmen along the aqueduct visit each station daily, note the current, voltage and power consumption on log sheets and in the case of the motor-generator sets check up on bearings for heating and observe the condition of commutators. Any irregularities are reported immediately.

Potential measurements are made periodically, between the aqueduct and earth by means of the $\frac{3}{4}$ inch ground test electrodes installed for the preliminary tests. These potential values are relative and are not conclusive evidence of corrosion control but by comparison with previous observations do indicate any changes in condition of pipe coating or anodes that may have taken place.

The most reliable indicator of the extent of corrosion control is the loss of weight of buried coupons. Several months are required to obtain satisfactory results. Forty-five coupon installations were made about the time the cathodic stations were started. Coupons should be of metal of the same composition as the pipe on which they are used. The first coupons installed on the Mokelumne Aqueduct were from a section of one of the plates used in fabricating the pipe, which is one half inch thick. It was found more convenient in weighing to use lighter coupons, and $\frac{1}{8}$ inch plates of the same kind of steel were substituted. A standard of size 3 inches x 6 inches was selected

for the coupons. All coupons are prepared by rounding the corners and edges with a grinder, and sandblasting. Each coupon is stamped with a designating number and weighed to the nearest 0.05 gram. Between the time of weighing and installation the coupons are protected from rusting by a film of oil which is washed off with gasoline just before placing in the soil. Locations where tests indicated the most severe corrosion were selected for observation. The coupons are buried in pairs at spring line depth and about two feet from the pipe. One coupon of each pair is attached to the pipe by means of an insulated copper conductor and an angle clip welded in place, and

FIG. 5. TYPICAL TEST COUPONS AFTER EXPOSURE FOR 620 DAYS

Top coupon of each pair attached to pipe and lower coupon free. Discoloration of free coupon does not represent corrosion. The attached coupons lost 0.6 grams per square foot per year, and coupon 168 lost 57.4 grams and 185 lost 68.8 grams per square foot per year.

the other is unattached. For convenience these coupons are referred to as "attached" and "free." When coupons have been buried for about a year they are removed, cleaned and weighed. The cleaning process known as "pickling" consists of suspending the coupons in a 1/10 normal sulphuric acid bath in which a lead anode is likewise suspended. A 6 volt storage battery is connected with the positive pole to the lead and the negative pole to the coupons. In this manner the oxides are removed by the hydrogen collecting at the cathodes. Wire brushes or abrasives should not be used in cleaning coupons. The attached coupon will corrode at the same rate as the pipe line

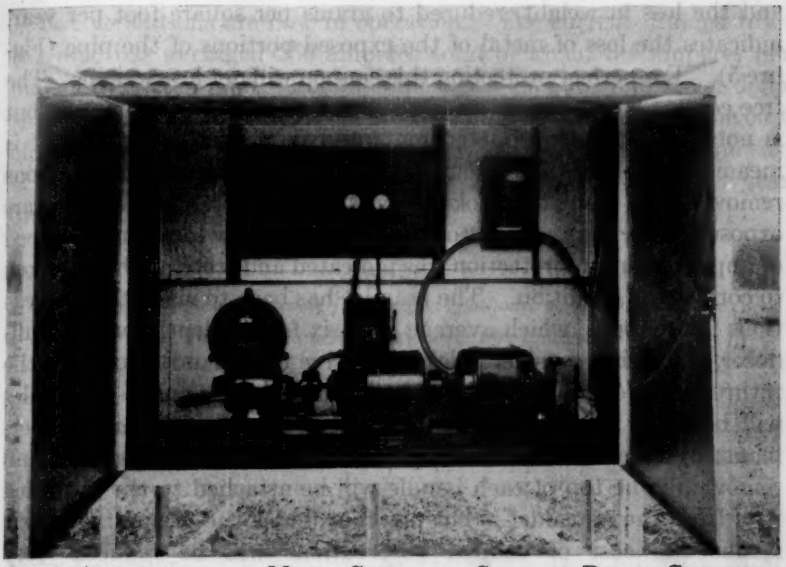


FIG. 6. ARRANGEMENT OF MOTOR-GENERATOR SET WITH DIRECT CONNECTED EXCITER

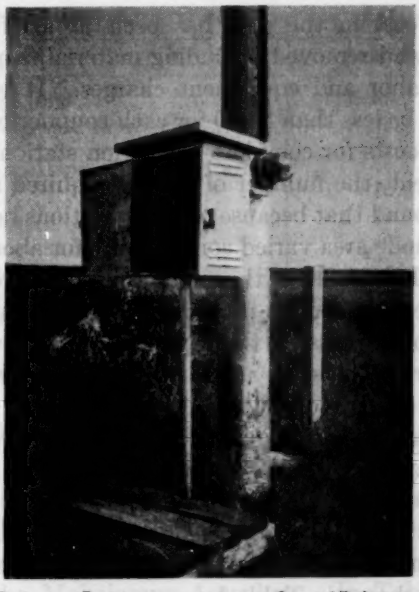


FIG. 7. TYPICAL INSTALLATION FOR 6 TO 15 AMPERE UNITS

and the loss in weight, reduced to grams per square foot per year, indicates the loss of metal of the exposed portions of the pipe (Figure 5). For perfect protection this coupon will not lose weight. The free coupon is not protected and will corrode due to local action but is not subject to long line currents and its actual loss of weight is meaningless. The free coupon is merely interesting. Coupons removed from along the Mokelumne Aqueduct after one to two years exposure show very satisfactory results. In a few locations the need of improvement in protection was indicated and steps are being taken to correct the condition. The practice has been to make the excavations for coupons, which average over six feet in depth, with a bulldozer, this being considered less expensive than hand labor. In the future it is proposed to install "attached" coupons only. The holes will be bored with post hole augers. Insulated pipe handles permanently attached to the coupons will be used for insertion and removal. The top of each handle will be attached to the insulated wire from the aqueduct about one foot under the ground. By this means the cost of coupon observations will be reduced very materially.

COSTS

The average cost in the past has been $20.00 for each pair of coupons buried and removed, including material, supervision, cleaning, weighing, labor and equipment charges. It is expected that future costs will be less than $5.00 for each coupon test.

Construction costs for complete protection stations vary with the type of equipment, the number of anodes required and local conditions. It was found that because of soil variations between locations the necessary anode area varied considerably for about equal current requirements. Inasmuch as the magnitude of current alone determines the degree of protection provided, it is of advantage to maintain the voltage as low as possible to keep power costs down. The anode life varies inversely in some undetermined manner as the current density. Both current density on the anode and voltage will be reduced by increasing the anode area. The average cost of 97 anodes, both pipe and carbon, was about $50.00 each.

Along the Mokelumne Aqueduct advantage was taken, in some instances, of existing structures to house the rectifier equipment. In cases where no structures are available a complete station can be erected at costs shown in Table I.

Twelve stations are now in operation. The current drained from the pipe has averaged 250 amperes total or about 3.1 amperes per mile of protected pipe. For the pipe in the San Joaquin River delta the current drained varies from 15 to 20 amperes per mile. The cost of power required to boost this drainage varies according to the size of the station but the cost has averaged $6.00 per ampere per year drained or $19.13 per mile.

TABLE I

Average construction costs for complete cathodic protection stations

(Investigation and test costs not included)

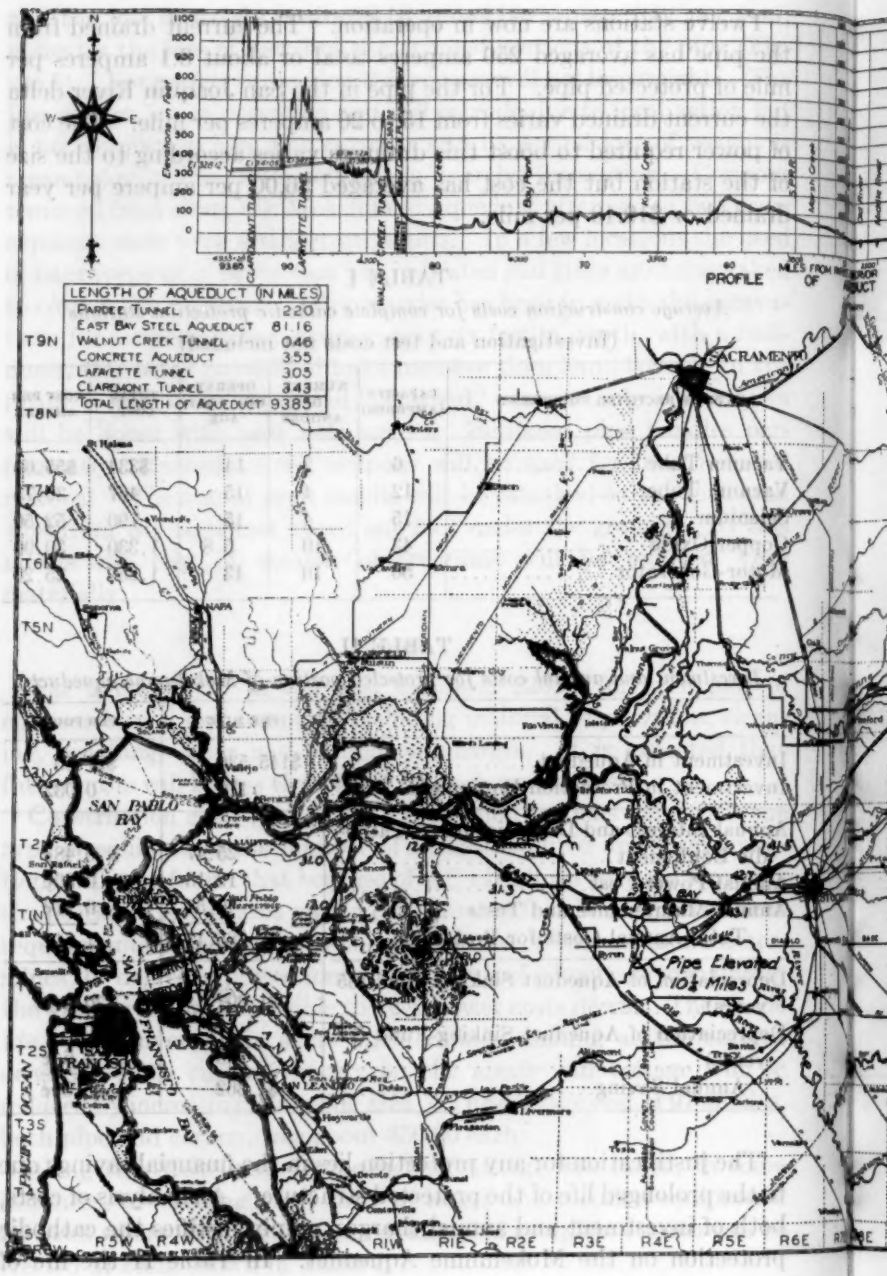
TYPE OF RECTIFIER EQUIPMENT	CAPACITY (AMPERES)	NUMBER OF ANODES	OPERATING VOLTAGE	TOTAL COST	COST PER AMPERE
Vacuum Tube	6	2	14	$334	$55.60
Vacuum Tube	12	4	15	467	39.75
Selenium	15	4	15	790	52.50
Copper Oxide	43	10	5.8	1,330	30.90
Motor-Generator	50	10	13	1,260	25.20

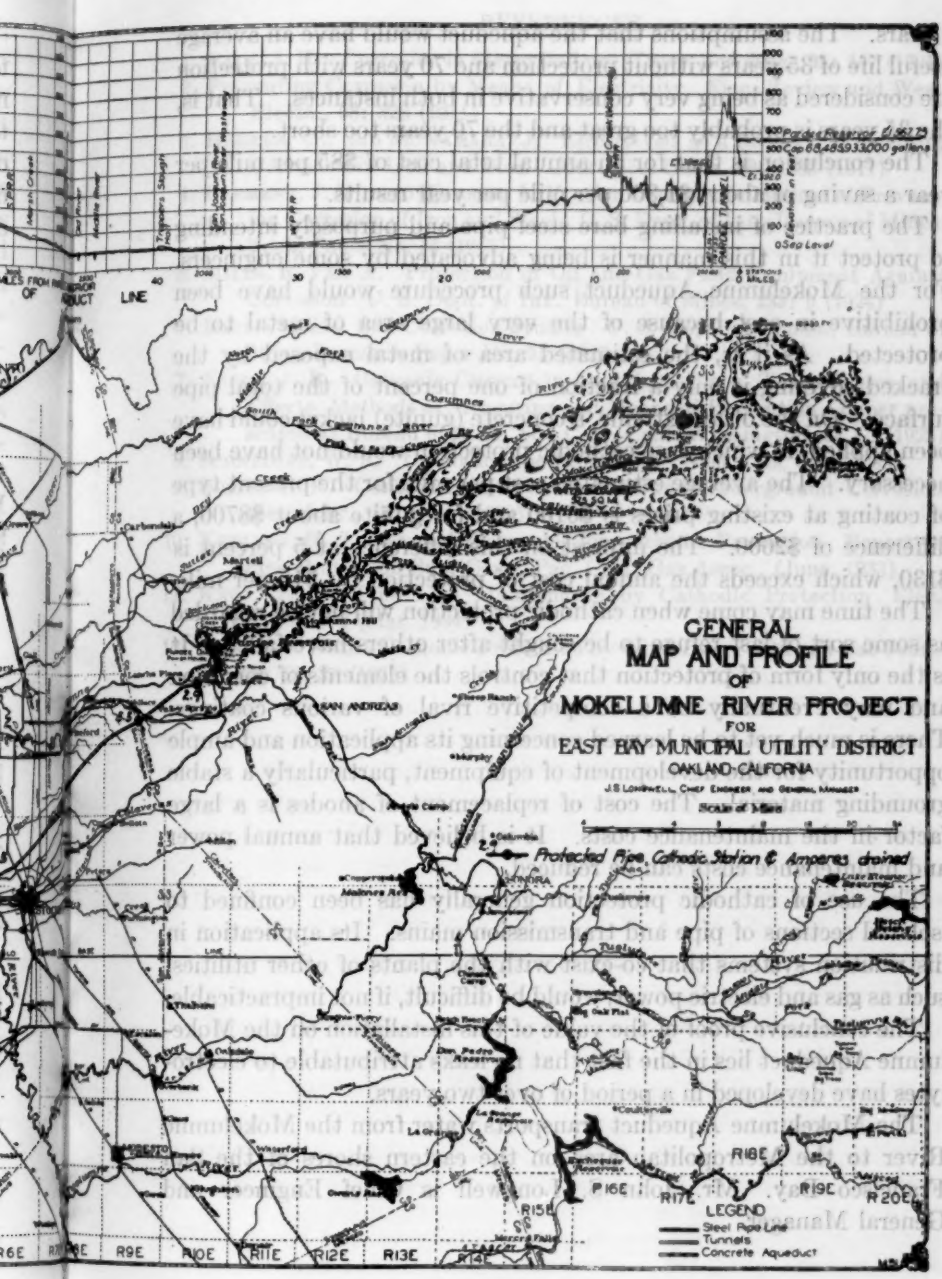
TABLE II

Investment and annual costs for protected portion of Mokelumne aqueduct

	PER MILE	PER FOOT
Investment in Aqueduct	$145,536	$27.56
Investment in Protection Equipment	170	0.032
Annual Interest and Depreciation on Cathodic Equipment	25.47	0.48¢
Annual Power Cost	19.13	0.36¢
Annual Maintenance and Tests	40.51	0.77¢
Total Annual Costs for Protection	$85.11	1.61¢
Depreciation of Aqueduct Sinking Fund (35 years)	$1,607	30.44¢
Depreciation of Aqueduct Sinking Fund (70 years)	245	4.64¢
Annual Saving	$1,362	25.80¢

The justification for any protection lies in the financial savings due to the prolonged life of the protected structure. An analysis of costs, both of investment and annual charges, amply justifies the cathodic protection on the Mokelumne Aqueduct. In Table II the life of rectifying equipment has been taken at 15 years and of anodes at


PROFILE
LENGTH OF AQUEDUCT (IN MILES)
PARDEE TUNNEL 2.20
EAST BAY STEEL AQUEDUCT 81.16
WALNUT CREEK TUNNEL 0.46
CONCRETE AQUEDUCT 3.55
LAFAYETTE TUNNEL 3.05
CLAREMONT TUNNEL 3.43
TOTAL LENGTH OF AQUEDUCT 93.85
SACRAMENTO
SAN PABLO BAY
SAN FRANCISCO BAY
OAKLAND
ALAMEDA
Pipe Elevated 10½ Miles
T9N
T8N
T7N
T6N
T5N
T4N
T3N
T2N
T1N
T1S
T2S
T3S
T4S
T5S
R5W
R4W
R1W
R1E
R2E
R3E
R4E
R5E
R6E


GENERAL
MAP AND PROFILE
OF
MOKELUMNE RIVER PROJECT
FOR
EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND-CALIFORNIA
Protected Pipe, Cathodic Station & Amperes drained
LEGEND
Steel Pipe Line
Tunnels
Concrete Aqueduct
SAN ANDREAS
R6E
R9E
R10E
R11E
R12E
R13E
R14E
R15E
R16E
R17E
R18E
R19E
R20E

5 years. The assumptions that the aqueduct would have an average useful life of 35 years without protection and 70 years with protection are considered as being very conservative in both instances. That is, the 35 years is probably too great and the 70 years too short.

The conclusion is that for an annual total cost of $85 per mile per year a saving of about $1,300 per mile per year results.

The practice of installing bare steel pipe and purposely intending to protect it in this manner is being advocated by some engineers. For the Mokelumne Aqueduct such procedure would have been prohibitive in cost because of the very large area of metal to be protected. As it is, the estimated area of metal exposed by the cracked covering is only a fraction of one percent of the total pipe surface. On the other extreme, a concrete (gunite) jacket could have been applied, in which case cathodic protection would not have been necessary. The average estimated cost per mile for the present type of coating at existing prices is $6100 and for gunite about $8700, a difference of $2600. The interest on this difference at 5 percent is $130, which exceeds the annual cost of protection by $45 per mile.

The time may come when cathodic protection will not be regarded as some sort of last refuge to be sought after others have failed. It is the only form of protection that controls the elements of corrosion and may eventually be a competitive rival of various coatings. There is much yet to be learned concerning its application and ample opportunity for the development of equipment, particularly a stable grounding material. The cost of replacement of anodes is a large factor in the maintenance costs. It is believed that annual power and maintenance costs can be reduced.

The use of cathodic protection generally has been confined to isolated sections of pipe and transmission mains. Its application in distribution systems that co-exist with the plants of other utilities, such as gas and electric power, would be difficult, if not impracticable.

The conclusive proof of the value of this installation on the Mokelumne Aqueduct lies in the fact that no leaks attributable to electrolytes have developed in a period of over two years.

The Mokelumne Aqueduct transports water from the Mokelumne River to the Metropolitan area on the eastern shores of the San Francisco Bay. Mr. John S. Longwell is Chief Engineer and General Manager.

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THE BIRMINGHAM INDUSTRIAL WATER SUPPLY*

By A. Clinton Decker

(Member of Engineering Commission, Birmingham Water Supply, Birmingham, Alabama)

A city of major industrial importance located in a narrow valley with only a negligible water shed area above the city and with streams of only minor importance flowing through the city and its adjacent areas—such a city is Birmingham, Alabama. As the city has grown in industrial importance the problem of a sufficient quantity of water of high quality for industrial purposes has become increasingly acute. The streams are being used to their capacity by industries which discharge their wastes back into them. For the greater part of their length they are grossly polluted with industrial wastes and therefore yield a very inferior quality of water to most consumers. Underground sources of supply have also been developed to their capacity. In addition to the fact that water from this source is extremely hard, its dependability is ever doubtful because of the presence of faults and fissures in the limestone formation which underlies the entire Birmingham Manufacturing District. For this reason studies looking toward the development of a water supply system to meet the present and future industrial requirements of the city otherwise blessed with vast natural resources, particularly iron ore, coal and limestone, were commenced in 1928.

The opportunity to obtain federal aid in the construction of an industrial water supply system, for which preliminary plans had been prepared, presented itself, and the city of Birmingham submitted an application in December 1933, to the Federal Emergency Administration of Public Works for a loan and grant with which to build a system having a normal capacity of sixty million gallons per day of high quality water for industrial purposes.

On January 14, 1936, final approval was obtained for funds with which to construct the project above referred to at a total esti-

* Presented at the joint session of the Southeastern and Kentucky-Tennessee Sections, April, 1937. Text revised by author in December, 1937.

mated cost of $5,808,000. The Works Progress Administration had approved a grant of $2,378,000, with which sum they were to undertake to construct certain definite portions of the project and the Public Works Administration had approved a loan of $3,430,000, with which certain other definite portions of the work were to be done.

As soon as the project had been approved and funds made available, the City Commission appointed an Engineering Commission to handle all matters in connection with the design and construction of the project. The members of this Commission are J. D. Webb, City Engineer who was designated as Chairman, O. G. Thurlow and A. Clinton Decker. The Engineering Commission selected A. C. Polk as Executive Engineer and following his selection an engineering organization was assembled as rapidly as possible.

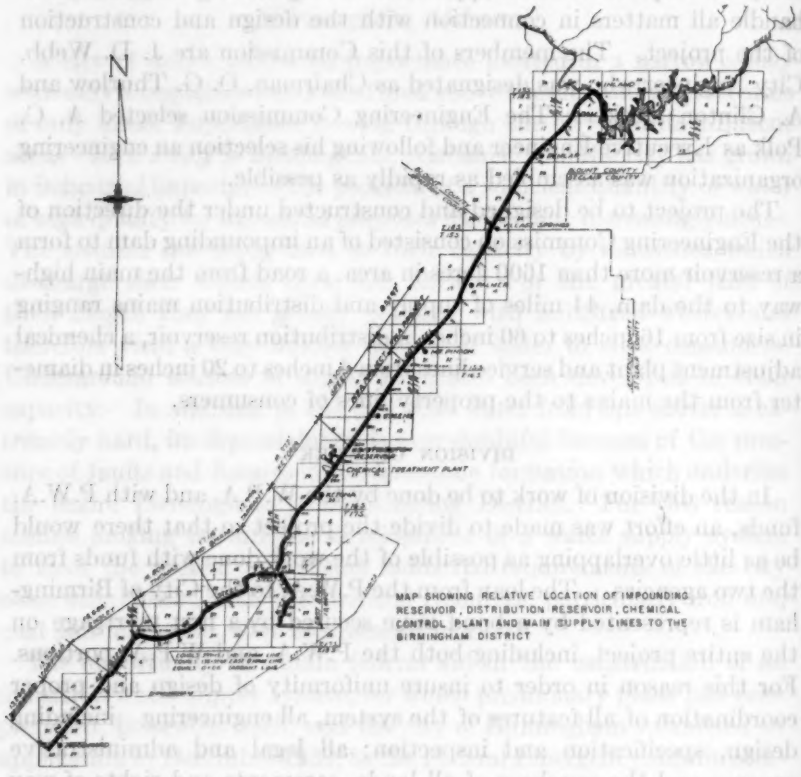
The project to be designed and constructed under the direction of the Engineering Commission consisted of an impounding dam to form a reservoir more than 1600 acres in area, a road from the main highway to the dam, 44 miles of supply and distribution mains ranging in size from 16 inches to 60 inches, a distribution reservoir, a chemical adjustment plant and service lines from 4 inches to 20 inches in diameter from the mains to the property lines of consumers.

DIVISION OF WORK

In the division of work to be done by the W.P.A. and with P.W.A. funds, an effort was made to divide the project so that there would be as little overlapping as possible of the work done with funds from the two agencies. The loan from the P.W.A. to the City of Birmingham is represented by a bond issue secured by a first mortgage on the entire project, including both the P.W.A. and W.P.A. portions. For this reason in order to insure uniformity of design and proper coordination of all features of the system, all engineering—including design, specification and inspection; all legal and administrative expense and the purchase of all lands, easements and rights-of-way were to be done with P.W.A. funds. In addition to these items P.W.A. funds were to be used for the construction of the impounding dam and for the supply main consisting of 86,000 feet of 60-inch pipe. The W.P.A. portion of the work consisted of clearing the flooded area of the impounding reservoir, the clearing, grubbing and excavation of a fixed portion of the impounding dam site, the construction of the road from the main highway to the dam site, the construction

of the distribution reservoir, 28 miles of distribution mains from 16 inches to 60 inches in diameter, a chemical adjustment plant and all the service lines.

In cooperation with the Federal Agencies involved, the work was undertaken in such a way as to give the maximum of direct employment within the shortest possible time. An immediate start was made on field surveys and the preparation of specifications for the



MAP SHOWING RELATIVE LOCATION OF IMPOUNDING RESERVOIR, DISTRIBUTION RESERVOIR, CHEMICAL CONTROL PLANT AND MAIN SUPPLY LINES TO THE BIRMINGHAM DISTRICT

24-inch cast iron mains for a total length of approximately five miles. A substantial portion of these mains were to be located in city streets while more than 90 percent of the remainder were located on the right-of-way of the Louisville-Nashville Railroad Company. Mr. T. E. Brooks, Vice-President of the railroad came to Birmingham at the request of the city, went over the general location of the mains of the entire project with the Engineering Commission and the

Executive Engineer and agreed to grant to the City an easement on the Railroad Company's right-of-way which included about 28 miles or 63 percent of the total of 44 miles involved in the construction of all mains. This right-of-way was granted for the sum of $1.00. In addition to this right-of-way about 22 percent is located in city streets or county highways. It was therefore necessary to purchase only about 15 percent of the rights-of-way necessary for the construction of the main lines.

It was necessary to purchase approximately 4,000 acres of land for the impounding dam and reservoir in order to secure the 1600 acres which will be flooded by the reservoir. Small acreage was also purchased for the distribution reservoir. Prevailing prices were paid for the land purchased and in only a few cases has it been necessary to institute condemnation proceedings in order to obtain a portion of the land needed. All land purchases, involving more than one-hundred separate transactions, have been handled by the City's Land Agent.

CONSTRUCTION PROGRESS

The next portion of the work undertaken, after the cast iron mains were started, was the clearing and preparation of the dam site and the clearing of the flooded area of the impounding reservoir. In less than sixty days after funds became available there were approximately 3000 men at work on the W.P.A. portion of the project; and subsequently there have been from 500 to 1500 men at work on the P.W.A. portion of the project.

Specifications for the 60-inch, 54-inch, 48-inch and 42-inch pipe provided for the use of welded steel pipe for all of these sizes with an alternate which permitted bids on 42-inch high tensile cast iron. Specifications permitted the fabrication of steel pipe in lengths from 38 feet to 48 feet at the option of the fabricator and providing that the fabricator should include in his per-foot price all specials and the necessary number of sleeve type couplings. The alternate for cast iron provided for delivery of 12 foot lengths of high tensile cast iron pipe including sufficient jointing materials to lay the pipe. The low bidder on this job was Chicago Bridge and Iron Company and Ingalls Iron Works Company, both of Birmingham, who submitted a joint bid. Their bid was not only the lowest of all bids submitted for furnishing steel pipe but was also lower in the case of the 42-inch than the bid submitted for furnishing of high tensile cast iron in 12-

foot lengths. Some of the steel pipe furnished under this contract was 40 feet in length but the major portion of the pipe was 48 feet in length—the longest lengths ever used on a major water works project in this country. After fabrication, all pipe was thoroughly

PORTION OF 60-INCH STEEL MAIN BEING LAID IN ROCK

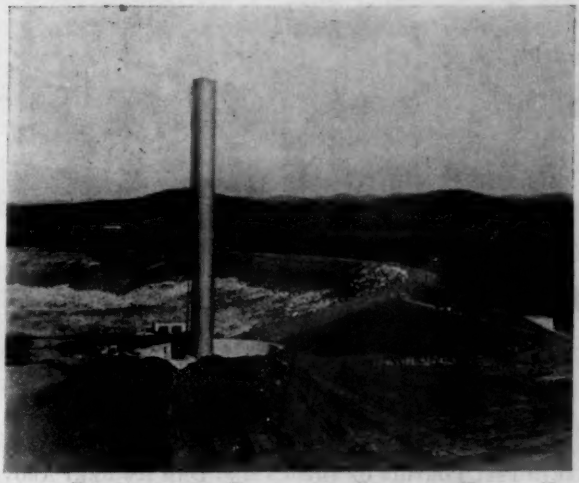
60-INCH STEEL MAIN NEAR CHEMICAL CONTROL PLANT

cleaned and coated and lined with Bitumastic Enamel. The lining was applied centrifugally with the pipe spinning at a rapid rate. The coating was applied while the pipe was rotated slowly.

Subsequent to the starting of the W.P.A. portion of the work, specifications were prepared and contract let for the furnishing of approxi-

mately 16 miles of 60-inch fabricated steel pipe for the P.W.A. part of the main. As in the case of the W.P.A. pipe contract, the Chicago Bridge and Iron Company and the Ingalls Iron Works Company, bidding jointly, were low bidders at $983,840 and thus received the contract.

All of the pipe has been delivered and laid. After the pipe was in place both automatic and manually controlled air valves were placed in the line. On the W.P.A. portion of the line specials for the air and blow-off valves were placed after the pipe had been laid because detail surveys necessary to fix the exact locations had not been completed when the order for this portion of the pipe was placed.



LOWER END OF DISTRIBUTION RESERVOIR SHOWING EARTH EMBANKMENT WITH PORTION OF RIP RAP PLACED AND SURGE TANK IN FOREGROUND

Such surveys had been completed when the P.W.A. portion of the pipe was ordered and the air and blow-off connections were made in the shop.

CONSTRUCTION OF DAM

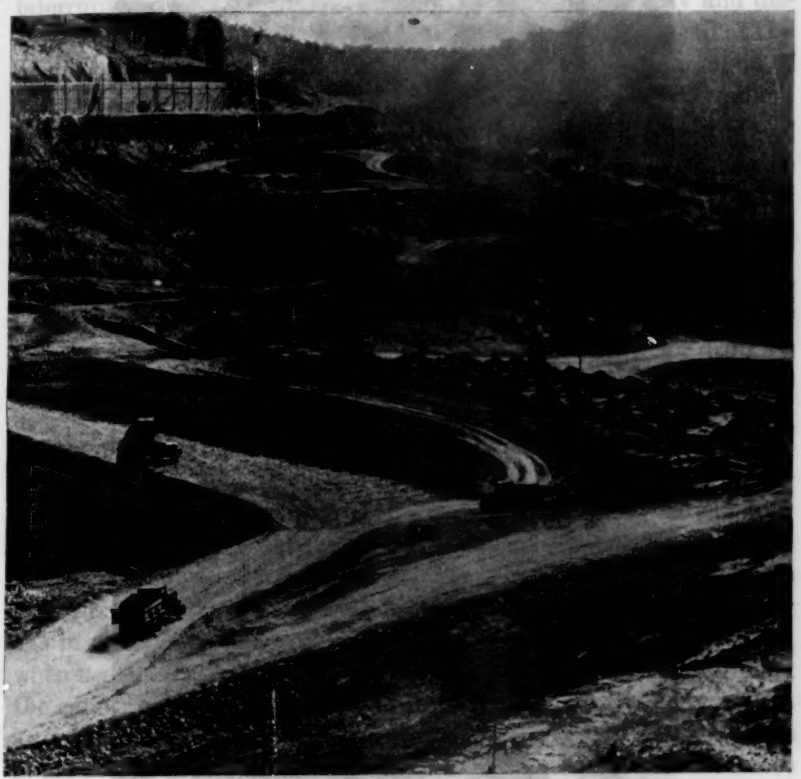
The preliminary surveys and studies had contemplated the construction of a gravity type concrete dam on the Blackburn Fork of the Black Warrior River near Inland, Alabama. These surveys had indicated that the foundations were of sandstone and shales of the coal age and that suitable foundations would be found at relatively shallow depths. Further investigations, after money for the project

had been made available, revealed the fact that a dam involving approximately 50 percent more concrete than had originally been contemplated would be necessary. It was therefore decided to make studies and designs for a rock-fill dam with compacted clay face. When the designs were completed, bids were requested on both types. The low bid for the rock-fill dam, $1,685,200, was more than $600,000 lower than the low bid on the concrete dam. The rock-fill dam was therefore selected and contract signed with Walsh Construction Company of Davenport, Iowa, for its construction. This dam is 195 feet high and 1100 feet long at the crest. There will be approximately 1,650,000 cu. yds. of material used in the construction. A

TOP OF EARTH FILL PORTION OF IMPOUNDING DAM 100 FEET ABOVE STREAM LEVEL WITH INTAKE TOWER IN BACKGROUND

diversion tunnel approximately 20 feet in diameter is being used to divert the stream during the construction of the dam and a portion of this tunnel will be used later as a spillway tunnel, the water entering through a shaft, when the reservoir is filled. The drilling and concrete lining of the tunnel and shaft are completed. In addition to this spillway which is designed to take care of a probable maximum flood with a large factor of safety, there is also provided an emergency spillway which will provide for the so-called five hundred year flood. This spillway consists of a concrete weir at elevation 780 with a ten foot earth fill above this. Should the water in the reservoir top this earth it will wash out and this spillway will

operate under a ten foot head, emptying the reservoir to a level four feet below normal which is elevation 784. Work on placing of stone in the dam is approximately 46 percent complete; and the clay fill is approximately 43 percent complete. This entire contract is approximately 64 percent complete.



VIEW SHOWING PORTION OF ROCK SECTION AND THE CLAY FILL SECTION OF DAM WITH SPILLWAY IN BACKGROUND

Under the terms of the contract with the Walsh Construction Company the completion date for the dam is March 19, 1938. However, there is a penalty and bonus clause which provides a penalty of $400.00 per day for each day subsequent to that date at which the dam is turned over to the city and a bonus of $400.00 a day for each day prior to that date at which the dam is completed with a maximum of $36,000. Indications now are that the dam will be completed during the month of March 1938.

The contract for the laying of the P.W.A. portion of the 60-inch steel main was awarded to Christie Hutchinson and Burton Company

PORTION OF QUARRY AT THE IMPOUNDING DAM

VIEW SHOWING UPSTREAM PORTAL OF TUNNEL IN FOREGROUND, IMPOUNDING DAM AT LEFT AND SPILLWAY IN BACKGROUND

of Birmingham, Alabama, for the sum of $269,508. They commenced work in September 1936, and completed laying the pipe, placing all valves and appurtenances, cleaning and testing the pipe

and all miscellaneous work in connection with their contract October 2, 1937.

The distribution reservoir is located near Ketona, a short distance north of the city. This reservoir is at such elevation that a full supply of water will be available in the district should there be any interruption in service in the upper portions of the supply and distribution system and will also be of value in maintaining head at the plants of consumers during periods of peak load. The amount of water available from this reservoir is approximately 65 million gallons. The work on this unit is 98 percent complete.

CHEMICAL TREATMENT

The chemical control plant is located immediately south of the distribution reservoir and is designed to provide both chlorination and pH adjustment. Although this system is to provide water for industrial purposes there will exist a possibility of employees in the industries inadvertently drawing water from this supply for drinking purposes. Cooperating with the State Board of Health it was decided to treat the water in a manner which would make it distinctly disagreeable and thus warn anyone who might use it by mistake that they had taken water from the wrong source. Superchlorination was therefore decided upon as being a satisfactory method of giving the warning above referred to, while at the same time sterilizing the water; so that if swallowed by an individual there would be protection against water borne disease. The chlorination has in addition to the Public Health value, the special advantage, when used for boiler purposes, of saving fuel as a result of improving the condenser vacuum. Where the vacuum is not as nearly perfect as it is possible to make it a considerable amount of money is spent in maintenance and cleaning condenser tubes and in addition thereto a considerable sum for fuel to steam out the tubes. In recent years several companies have used chlorine for the purpose of controlling the film of slime which grows in the tubes. This expense will be saved to users of the water from this system since it will have been chlorinated before delivery to the consumers. Preliminary analyses had indicated that the water has a low pH during certain portions of the year and it was decided to install the necessary dry feed equipment to apply chemicals to adjust the pH. Chlorination equipment and dry feed machines are both to be automatically operated.

Two typical analyses of the water are as follows:

	PARTS PER MILLION	
	April 1937	Oct. 1937
Silica (SiO_2)	3.4	6.6
Iron (Fe)	0.5	0.6
Calcium (Ca)	2.0	2.2
Magnesium (Mg)	0.8	1.0
Sodium (Na)	2.8	2.0
Sulphates (SO_4)	3.5	3.2
Carbonates (CO_3)	4.8	4.8
Chlorine (Cl)	2.0	2.0
pH	6.9	7.0
Solids in Suspension	20.6	52.0

The chemical control plant is approximately 75 per cent complete. The road is completed. The W.P.A. excavation at the dam site is completed and the clearing of the more than 1600 acres of the reservoir area is approximately 92 percent complete. All of the cast iron mains have been laid and work has been completed on approximately 80 per cent of the 4-inch to 20-inch service lines. All of the steel mains ranging in size from 42 inches to 60 inches in diameter have been laid and the placing of air and blow-off valves, cleaning and retouching of the inside of the pipe and the testing has progressed so that this pipe section of the project is 95 percent complete.

This system represents a new field of water supply activity. It seems probable that other areas where quantity of water for industrial purposes is limited will find this type of supply the answer to their industrial water problem.

THE RAW WATER SUPPLY OF THE FORD MOTOR COMPANY*

By William W. Dulmage

(*Superintendent of Power, Ford Motor Co., Dearborn, Mich.*)

In order to appreciate the present method of handling of mill water at the River Rouge Plant of the Ford Motor Co. let us delve into the past and see what manner of events built up the present system with its tremendous capacity for clean water at a temperature better suited for efficient condenser cooling and general service than had heretofore been the case. To explain why this is so requires that the old mill water system be briefly sketched.

Along the southerly side of the Rouge Plant area runs the River Rouge, which winds in a general eastward direction to empty into the Detroit River. When the Ford Motor Co. first began establishing itself on its present site this was little more than a sluggish creek with a normal summer temperature of from 75° to 80°F. Subsequently the stream bed was dredged to a uniform depth of approximately 20 feet for the admittance of large boats from the Detroit River directly to the Rouge Plant. Inland at the edge of the plant a turning basin for shipping was provided; and, leading from this basin, and almost perpendicular to the stream's course, a boat slip was excavated. This slip is 250 feet wide by a half a mile in length. It is from the closed or upper end of this that water was drawn for plant and mill use. From the intake it passed through a tunnel to a screen house. The intake draft averaged 567,648,000 gallons every 24 hours. Of this total, 20 percent was for plant use and 80 percent was for the power house. The latter supply then passed through a tunnel 12.67 feet in diameter and approximately 5000 feet long. After passing through the condensers, the water traveled through an outlet tunnel or tail race, the mouth of which emptied into the Rouge stream a little below the turning basin. On the other hand, water from the steel mill, glass plant etc., following its use for cooling

* Presented before the Central States section, August, 1937.

and slushing, was also returned to the River Rouge with the result that during summer months the stream's temperature rose to between 84 and 87 degrees F. However, that increase in temperature, over normal, presented only a part of the difficulty.

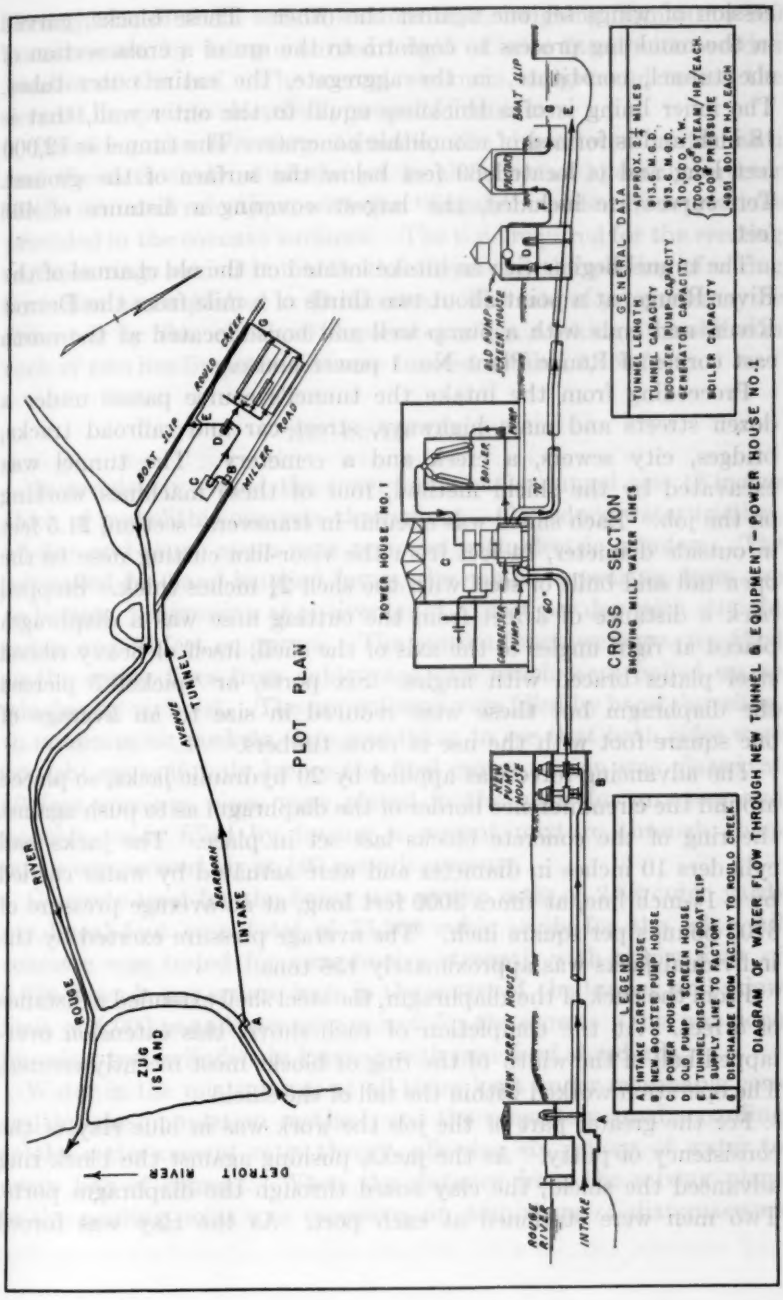
The chief problem, now corrected by the present tunnel, lay in the fact that the entire plant requirement was approximately four times in excess of the total Rouge River flowage. The average flow of the Rouge, normally available to the boat slip at that time, was 200 cubic feet per second. But since the amount of water drawn from the upper end of the slip approximated 900 cubic feet per second, it follows that 700 cubic feet of this had to be obtained by reversing the current. This resulted in the creation of a loop composed of an immense quantity of water that was actually used over and over again. That is, all of it except the portion which broke away to continue onward to join the Detroit River. The result of this recirculation, from the slip to the plant, from the plant back to the stream channel, and then again to the plant etc., tended to build the temperature of the water up to an average of 90°F. and there have been summer days when it reached a maximum of 95°F.

Now with the new tunnel in operation, the intake water will average 71° for the hottest months; 65° for the next hottest months and so range downward to a winter minimum of around 33°. Furthermore, by using the old intake tunnel for the expulsion of used waters, both the boat slip and the Rouge River will undergo a continuous flushing and thereby relieve the turbid condition of the water which then existed.

THE NEW WATER TUNNEL

Construction of the tunnel, which was to accomplish this end, was begun August 15, 1929. Its exact daily capacity is 913,600,000 gallons—amounting annually 334,464,000,000 gallons, or over 80,000,000,000 more than is used in a like period by the cities of Detroit, Cincinnati, Washington, D. C. and Philadelphia combined. So far as known it is the largest tunnel ever built to serve the needs of a single industrial concern.

Generally described, the tunnel is a true cylinder with an inside diameter of 15 feet and an outside diameter of 21 feet. Two thicknesses make up its walls. The outer is composed of interlocking cement blocks, weighing 2,420 pounds each, ten to a ring, the suc-


PLOT PLAN
ZUG ISLAND
ROUGE RIVER
DEARBORN INTAKE AVENUE TUNNEL
MILLER ROAD
ROUGE CREEK
BOAT SLIP
DETROIT RIVER
CROSS SECTION
SHOWING MILL WATER LINES
POWER HOUSE NO. 1
BOILER
PUMP
CONDENSER PUMP
NEW PUMP HOUSE
NEW SCREEN HOUSE
OLD PUMP & SCREEN HOUSE
INTAKE
ROUGE RIVER
GENERAL DATA
TUNNEL LENGTH APPROX. 2¼ MILES
TUNNEL CAPACITY 913.6 M.G.D.
BOOSTER PUMP CAPACITY 913.6 M.G.D.
GENERATOR CAPACITY 110,000 K.W.
700,000 STEAM/HR/EACH.
BOILER CAPACITY 1400 @ PRESSURE
4951 BOILER H.P. EACH
LEGEND
A INTAKE SCREEN HOUSE
B NEW BOOSTER PUMP HOUSE
C POWER HOUSE NO. 1
D OLD PUMP & SCREEN HOUSE
E TUNNEL DISCHARGE TO BOAT SLIP
F SUPPLY LINE TO FACTORY
G DISCHARGE FROM FACTORY TO ROUGE CREEK
DIAGRAM OF WATER FLOW THROUGH NEW TUNNEL & EQUIPMENT – POWER HOUSE NO. 1

cession of wings set one against the other. These blocks, curved in the moulding process to conform to the arc of a cross section of the tunnel, constitute, in the aggregate, the entire outer tubes. The inner lining is of a thickness equal to the outer wall, that is 18 inch and is formed of monolithic concrete. The tunnel is 12,000 feet long and is located 60 feet below the surface of the ground. Ten curves are included, the largest covering a distance of 466 feet.

The tunnel begins with an intake located on the old channel of the River Rouge, at a point about two thirds of a mile from the Detroit River, and ends with a pump well and house located at the north east corner of Rouge Plant No. 1 power house.

Proceeding from the intake the tunnel's course passes under a dozen streets and main highways, street car and railroad tracks, bridges, city sewers, a creek and a cemetery. The tunnel was excavated by the shield method, four of these machines working on the job. Each shield was circular in transverse section, 21.5 feet in outside diameter, 14 feet from the visor-like cutting nose to the open tail and built of steel with the shell $2\frac{1}{4}$ inches thick. Stepped back a distance of 3 feet from the cutting nose was a diaphragm placed at right angles to the axis of the shell, itself of heavy ribbed steel plates braced with angles. Six ports, or "pickets" pierced the diaphragm but these were reduced in size to an average of one square foot with the use of cross timbers.

The advancing force was applied by 20 hydraulic jacks, so placed around the circumference border of the diaphragm as to push against the ring of the concrete blocks last set in place. The jacks had cylinders 10 inches in diameter and were actuated by water carried by a $1\frac{1}{2}$-inch line, at times 3000 feet long, at an average pressure of 3700 pounds per square inch. The average pressure exerted by the individual jacks was approximately 156 tons.

From the back of the diaphragm, the steel shell extended a distance of 5 feet. At the completion of each shove, this extension overlapped half of the width of the ring or blocks most recently erected. The operation worked within the tail of the shield.

For the greater part of the job the work was in blue clay of the consistency of putty. As the jacks, pushing against the block ring advanced the shield, the clay oozed through the diaphragm ports. Two men were stationed at each port. As the clay was forced

through one of them cut off sections of it, weighing from 50 to 100 pounds each, by using a curved length of wire fitted with wooden handles at the end. The second workman caught and tossed the severed lumps into electrically propelled tram cars.

Following the necessary advance, the jacks were withdrawn and the next circle of blocks set in place by an hydraulic erector of the arm and ram type, with the "fingers" inserted in depressions provided in the concave surfaces. The time required for the erecting of these rings varied but both the shove and the placing of a ring was often accomplished in 45 minutes. The rate of progress may be indicated by the month of September when working two shifts in each of two headings the distance covered totaled 2,160 feet.

THE INNER LINING

As previously stated the inner lining of the tunnel was 18 inches thick of monolithic concrete throughout. To reduce water friction, all fins and rough spots were removed with electric grinders. The job called for hand-handled forms, the complete pouring, from top to bottom, progressing at an average of 51 feet per day with straight going, and 40 feet on curves. The bottom forms were gravity filled to the spring lines from cubic yard cars which were pulled up on winches by air hoist. The upper forms were filled by hand shoveling, to minimize air pockets, care was taken to see that both sides were brought up uniformly before the final crown section was placed, at 10 foot spacings pipes were placed in the walls for grouting and possible voids filled by forcing a cement mixture through them under compressed air at 100 pounds pressure.

Concrete used for the lining was at the ratio of 2.88 cubic yards per lineal foot or a total of 33,560 cubic yards for the job. All concrete was tested for compressive strength with a low limit of 2,650 pounds per square inch in the invert of the tunnel, and a low limit of 4,000 pounds per square inch for the crown. This was done by using test cylinders in keeping with practiced standards.

Water in the mixture was at all times kept under reasonable control by the inundation method; and the concrete poured according to the water-cement ratio theory, allowing six gallons of water to every bag of cement. When the distance from the mixing plant to the pouring point was excessive, an admixture of diatomaceous

earth was used to prevent segregation. This was held to 2 percent for the longest haul of 3,200 feet.

THE TUNNEL INTAKE

In an earlier day the River Rouge, as first mentioned, took its way past the Ford Motor Company's Rouge Plant to the Detroit River, but, just before arriving at what seemed its logical destination, described a huge "S" curve to finally meet the stream around the nose of a peninsula. Subsequently a short channel was cut through the peninsula's neck. Thereupon, the peninsula became an island and that portion of the Rouge which skirted the inland edge of it found its current suddenly reversed. This operation is pertinent because it explains why the water, discharged from the Ford plant into the Upper Rouge stream under the present set up, will at no time come in conflict with the intake. Instead it will continue down to join the confluence of the reversed current and so pass through the short cut canal into the Detroit River.

It was because of this condition that a point on that portion of the channel in which the current flows inland was selected by Ford engineers for the intake tunnel. Work on this job was begun in December, 1929. It started with the building of a coffer dam. This consisted of a sheet piling back wall, 350 feet long and parallel to the harbor line with the ends of a crescent shaped wall attached to its either end. The bow at its greatest width, was 40 feet. These major walls were held together by diaphragm walls set perpendicular to the back wall, and forming pockets that were filled with clay. Following the excavations made possible by the arrangement, 1400 piles were driven as supports for the intake and screen house structures. The concrete foundations for the substructure began at a depth of 37 feet below the river level. Except for a raised walk, suggestive of a narrow dock, and which is flush with the harbor line, the work extending back to the facade of the screen house is all under ground. Longitudinally, this area is now traversed by side walks and that portion of a traveled highway which was temporarily closed during operations. Beneath the dock like platform is the intake gate gallery and, still deeper, the intake passages which extended under the street and screen house. The intake unit, thus completely covered, is 54 feet by 154 feet in area.

The screen house structure is 44 feet by 104 feet. It is brick, steel and concrete, floored with tile and contains all controls governing,

screens, gates, stop log, pumps, etc. It is served by a 10-ton overhead electric crane and heated by a small boiler station in an under floor room.

Back of the screen house, and also below grade, is the discharge chamber. From a base practically as wide as the screen is long, it swings gradually to the left, always narrowing, to resolve itself finally into the transition chambers which connect with the main tunnel.

The elevation of the water at the harbor line is approximately 94 feet above datum. The stream along the intake is dredged to afford a basin 350 feet long by 180 feet wide. This will supply a maximum stream depth of 35 feet. The bottom of the intake gates are about 32 feet below the level of the river.

The water passes through a series of 20 openings, each 5 feet x 10 feet in size at velocity of 1.4 foot per second. These openings are so arranged, by means of obliquely placed side walls, as to present inlets natural to the approaching current. But the alternate wall in each of these entrances is soon terminated, whereupon the water will progress toward the screen house through ten passages. In doing so it will move at a common flow level for a distance of approximately 63 feet, but under ceilings that gradually ascend. At the end of the stated distance the floor meets a wall, 14 feet high and perpendicular to it. This will form a barrier against the heavier silt. Lifting over this obstruction and given ample head room by reason of the higher ceilings, the water will pass through stationary trash racks to be freed of heavy debris such as drift wood, logs and ice. Continuing at this new floor level, and still divided by the 10 passages, it will move through the mesh of electrically operated traveling screens. Set behind these screens at a point where they rotate above the main floor, are spray nozzles. The rapid streams will clear the screens of that debris not caught by the trash racks and which the passing intake may have deposited on them. This will fall into a trough and be automatically carried away to a refuse sump. Briefly, this is the manner in which water, taken from a navigable stream at the rate of almost one billion gallons every 24 hours, will be led into and transformed to an underground stream without creating a current at the intake unfavorable to navigation. As to measurements over all and grades, the distance from the gates to the tunnel end of the transition chamber is approximately 265 feet.

The floor of the discharge chamber is level except for the last 42 feet in which distance it drops 11 feet. Then within the mouth of the tunnel, there begins an 8 degree vertical curve, 160 feet long and dropping 19 feet in that distance. From this last point there is a 5 foot drop, gradual, to the pump well located at the Rouge Plant Power House, some two miles away.

Water will flow through the tunnel with a velocity proportionate to the river elevation and constantly replace that which is pumped out. Assuming a 9,000,000 gallon consumption at the Power House and of the line, the velocity of flowage in the tunnel will be at the rate of 7.9 feet per second.

When it becomes necessary to clean any or several of the intake passages, the gates will be closed and debris and silt be removed in buckets operated by the overhead crane. In the screen house floor, and set immediately over the several trash trucks, are manhole gratings which will permit the descent of divers to remove troublesome objects that may become fast and so interfere with water flow.

THE PUMP WELL

Immediately adjacent to No. 1 power house, at the north end of the tunnel, is located the pump well. This was started in March 1930, and sunk by the open caisson method. It is approximately 65 feet deep, 70 feet in outside diameter and walled by circular rings reinforced concrete 4 feet thick. The bottom is floored by a mat, convexed on the under side, with a central thickness of 8 feet as against 3 feet 5 inches on the edge. The initial ring, 20 feet in depth, was provided with a cutting edge characteristic of open caisson practice. It and the succession of 5 feet rings, were sunk under their own weight. The descent being through blue clay, no notable resistance was encountered until the cutting nose reached a strata of hardpan at a depth of 80 feet. To augment the self contained pressure at this point, the uppermost ring was weighted by distributing 1200 tons of pig iron along its top. This was completely resisted by the hard pan; but since the imposed burden greatly exceeded the planned super structure, no more was asked.

With the sinking of the well came the construction of a circular tunnel, 15 feet in inside diameter and 116 feet long which curves

to meet the main tunnel. This segment, driven under 18 to 20 pounds air pressure, is of monolithic concrete. Directly opposite the point where it connects with the well, and departing from the well on the power house side a second tunnel segment, built of the same material and in the same manner, extends 168 feet to empty into a surge chamber. Between the two, and within the well, is the huge sluice chambers. At the main tunnel end of this chamber are two sluice gates provided with flap checks. In the normal process of water consumption the supply delivered by gravity from the intake, will fill submerged passages located near the floor of the tunnel at points just beyond the sluice gates. Within these passages it will be picked up by the booster pumps, lifted a height of 23 feet, discharged by them across sluice chambers to move through the 168 foot tunnel to the surge chamber. Should sudden surges occur, due to quick changes in demand, and the surge chambers contents lift beyond a certain level, it will rise against and open flaps which will permit the excess to spill into the tail race tunnel. When at week ends or shut downs or for any reason the booster pumps are inactive, the flap checks in the sluice gates will operate with the normal gravity delivery and the tunnel water merely rises to remain stationary at a level with that at the intake two miles distant.

The boosting is performed by four pumps. Each is of the vertical propeller type, with synchronous driving motors and control. The individual capacity will be 157,750 gallons per minute, against a 14.42 foot head. Each will have two operating speeds 277 R.P.M. and 138 R.P.M. The two named speeds will both be had from a single two speed motor. This and its control are arranged so that both motor and stator may be connected for 26 or 52 pole operation, these corresponding to the respective speeds of 277 and 138 R.P.M. The motor, at the named speeds, is rated at 700 and 350 horse power.

Some idea of the physical size of these pumps may be had from the fact that each, without valves, weighs 33 tons. The height of each, from the top of the valve to the shaft bearing, is 23 feet. The shafts measure 6 inches in diameter and 29 feet long. The two elbows with which each is provided, and which incidentally, were cast in the Rouge Plant Foundry, measure 7 feet at the suction end and 6 feet at the discharge. The discharge openings are fitted with 72 inch valves and the intake with 60 inch valves. The elbows are

crossed with vanes to prevent the water from whirling and so provide a constant direction upon exit. To lessen interference as between the water thrown out of the two forward openings and those immediately back of them, deflectors have been placed at the mouth of each pump discharge. These tend to preserve the column formation after leaving the pumps and so give each a kick in the desired direction.

Piezometer pipings, set in the walls and ridge of the tunnel linking with the surge chambers, make provision for determination of the amount of water moving. The pump house, containing the various controls, is of structural steel, concrete and brick with exterior walls which correspond in character with those of No. 1 power house. In size it measures 64 feet by 70 feet and is 48 feet high. The floor is serviced by a 20-ton overhead electric crane.

In making the various change overs near the pump well and beneath the power house, tunnels formerly used for intake were converted into tail races. The undertaking also necessitated the excavation of a tunnel of the boxed type 125 feet long. This connected with the surge chamber and extended under the power house. Both it and the chamber were handled within steel sheeting. To accomplish a part of this work, holes were cut in the power house floor and the tunnel driven both ways from the shafts.

LOW HEAD PUMPS

Water upon entering the power house for condenser cooling purposes pass under generators with pumps of the following capacities:

	NUMBER	MOTOR H.P.	G.P.M.	G.P.M., TOTAL CAPACITY
No. 1-2 Generator	2	335	61,500	123,000
No. 3-4 Generator	2	325	58,700	117,400
No. 7 Generator	2	500	30,000	60,000
No. 8 Generator	2	500	30,000	60,000
Blower No. 2	1	60	7,500	7,500
Blower No. 3	2	75	7,800	15,600
Blower No. 4	2	75	7,800	15,600
Total Low Head				399,100

HIGH HEAD PUMPS

Mill water for various buildings and general service are listed below:

80 pound service

LOCATION	PUMP TO	NUMBER	HORSE POWER	G.P.M.	TOTAL G.P.M.
No. 1 PH	Coke Ovens	3	500	7,500	22,500
No. 1 PH	Blast Furnace	3	450	7,000	21,000
No. 1 PH	Blast Furnace	1	350	7,000	7,000*
No. 1 PH	General Service	1	125	2,000	2,000
No. 1 PH	General Service	1	458	10,000	10,000
Screen House	General Service	5	700	10,000	50,000*
Screen House	General Service	1	800	11,000	11,000
Screen House	General Service	1	1,200	20,000	20,000*
No. 1 PH	Fire Pumps	2		1,500	*3,000*
No. 1 PH	Fire Pumps	2	2,500	37,500	75,000
Total High Head					221,500

(Pumps marked * are used for standby and are steam driven. Total g.p.m. = 77,000.)

CHLORINATION OF MILL WATER

To prevent slime formations on condensers, tubes, service water lines, strainers etc., throughout the plant we chlorinate the water at two points. First in the pump well where the residual is kept at 0.4 p.p.m. and at the screen house where a residual of 0.35 p.p.m. is maintained.

LICENSING OF PLANT OPERATORS AND SUPERINTENDENTS*

By Edward R. Stapley

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(*Oklahoma A. & M. College*)

So much has been written or said on the subject of Licensing that it would seem next to impossible to write or state anything new. Like the weather, it is a subject about which we can talk a lot but so far at least, many of us, or our states, have been able to accomplish little about it. Although doubtful that I can present to you anything new, I shall try to attack this subject somewhat differently than it seems to have been attacked by other speakers and writers. Repetition is often an effective method of emphasis, and continued pounding, it is said, wears away the toughest rock. However, I fear some of the obstructing boulders we most need to hammer are not present at this meeting.

So far nine states have licensing in some form. Only one of these nine, Texas, is included in this Southwest Section. In four of these, California, Georgia, South Carolina and Texas, the plan is on a voluntary basis. In one, Michigan, the plan was put into effect by a Regulation set up by its State Health Department. In four states, New Jersey, New York, Ohio and West Virginia, acts have been passed by the State Legislatures authorizing a licensing plan and setting up the general steps to be followed.

In practically all of these states three grades of licenses are issued. These are classed as Grades A, B, and C, or as 1st, 2nd and 3rd Class, and depend primarily on the type of plant or system. The qualifications as to training and experience vary. In some states the licenses are issued for life, while in others, they expire and must be renewed every one to three years. In one state, New York, chlorinating systems serving cities of over 40,000 population must employ a Class 1 operator. In some states the plan affects only publicly

* Presented before the Southwest Section, October, 1937.

owned plants and systems, while in others it regulates both public and privately owned public supplies.

In California, the licensing, on a voluntary basis and for members of the Section only, is the result of a Resolution adopted by the California Section of the A.W.W.A. on September 23, 1935. The first Certificates, 110 in number, were issued on January 1, 1937.

In Georgia, the voluntary licensing has been responsible for a very noticeable jump in the Short Course attendance, 200 attending the 1936 School.

New Jersey, the first state to adopt the plan of licensing, passed an Act to this effect February 9, 1918. Under this Act 84 "grandfather" licenses were issued. There are now nearly 1,000 licensed water and sewer men in the state, about one-third of this number being for water plants and systems, and about two-thirds for sewage treatment plants. Some cities have 10 or 12 licensed operators, only a small number of whom are actually operating plants. In order to remedy evils of this situation, an Amendment to the original Act is being sought to require annual renewal of licenses and set up a range of fees for original licenses and renewals. A new Tenure-of-Office Bill has also been submitted to the legislature but failed to pass. This bill would prevent the removal of any licensed employee with 5 years consecutive service except for good cause and after a public hearing, following the submission of written charges and notice of a hearing at least 5 days prior to said hearing. This tenure act will only affect employees of publicly owned systems, since the state cannot legally dictate the employment methods or policies of private concerns.

New York's bill, passed the legislature last spring, after 10 attempts, and became effective July 1, 1937. A voluntary system had been planned if the bill had failed to pass the last legislature. Provisions of the New York regulations which take into account the population of cities as well as types of plants have already been mentioned.

Licensing in Ohio also became effective by separate statute in July of this year. Previous to this time water plant operators and chemists were included in the Engineers' Registration Act.

In South Carolina, it is proposed to supersede their voluntary plan by an act or regulation when 75 percent of the operators in the state have become qualified for voluntary licenses.

In the Texas voluntary plan, all applicants must first apply and

qualify for the lowest grade, Grade C, and can only hold this license for a period of 3 years.

The West Virginia Act requires that a licensed man must be on duty at water purification plants during all shifts. A single licensed superintendent is not deemed sufficient for dependable operation twenty-four hours a day. Here written examinations are numbered to conceal the identity of applicant. Three independent gradings are made by the Department of Health, the Head of the Sanitary Engineering Department of the State University, and selected operators. These grades are weighted and then averaged. In this state about half the plants are privately owned. The Licensing Act, considered by some operators originally as a bugaboo, is now held in high favor. A coördinated training program for operators, promulgated by the University, has been considered largely responsible for the success of the law.

Colorado, Wisconsin, and Oklahoma have submitted Licensing Bills to one or more legislatures without success so far.

In Pennsylvania where a large number of the water systems are privately owned there seems to be a great deal of antagonism to licensing.

What is a license? According to the Dictionary, it is a document issued by the proper authorities granting legal permission or authority to do certain things that would otherwise be contrary to the law. Usually certain qualifications of training, experience, or both must be satisfied. It is in this sense it is considered in the application under discussion.

Just what individuals, groups, or governmental subdivisions are interested in, or would be affected by, a plan for the licensing of men charged with the supervision or operation of water systems, water purification and treatment plants, and sewage treatment plants? And, what are the particular advantages and disadvantages of such a licensing plan to these individuals and groups?

Those affected would include the Superintendents and Operators themselves, the Mayors and City Councils or Commissions and City Managers, the State Health Department and its subordinate units, and finally the Public.

Considering first the individuals to be licensed, the advantages and disadvantages might be listed as follows:

The case for—

1. To encourage men to improve themselves.
2. To stabilize employment by reducing personnel changes.

3. To increase respect of municipal officials for departments and plants.
4. To insure better salaries being paid.
5. To attract higher grade men to the positions.
6. To awaken more public interest in systems and plants.

The case against—

1. Too much red tape in making a living now.
2. Too much government regulation.
3. Extra expense for license and renewal fees when pay is already too small.
4. Danger of political control of licensing body.
5. Older men with little school training placed at a disadvantage in examinations.
6. Employees of private companies feel they have nothing to gain in the way of security or pay to balance extra expense and trouble.

Next take the Mayors', City Councils', Commissioners' and City Managers' viewpoint.

The case for—

1. Competent operating personnel guaranteed.
2. Superintendents, being responsible for systems and plants and quality of products, will also be certified as competent.
3. City will benefit from increased interest of employees in training.
4. Public health of community protected.
5. Better economy in use of city funds.
6. Better care and maintenance of plants and equipment.
7. Excuses furnished for refusing to employ job-seeking supporters.

The case against—

1. Superintendents should be executives, rather than schooled in sanitation, and boards cannot set up examinations to determine this quality.
2. Licensees will lose interest in continuous self improvement.
3. Security of position will tend toward inefficiency.
4. Interference with present Civil Service systems.
5. Restricts right to hire and fire. The Wisconsin Licensing Bill was reported as being killed in the legislature because "it would interfere with the prerogative of municipal officials to hire and fire employees."
6. Decrease opportunity to pay off political debts.

For the State Health Department the case seems to be rather one-sided according to the viewpoint of health department officials.

The case for—

1. Superintendents and operators would have at least a minimum knowledge of their jobs.
2. Continuity of plant operation.
3. Greater familiarity with Health Department requirements.
4. Increased control of personnel in plants for the safeness of whose products they are largely responsible.
5. Greater protection to public health.

The case against—

Apparently none.

Of all groups the general public is perhaps least interested, but should they be?

The case for—

1. Maximum protection for community health.
2. Increased potability of water supply.
3. Proper operation of city's investment in plants and equipment.
4. Sensible economy in expenditures from city funds.
5. Reduced tendency of irresponsible politicians to make political footballs of plants and systems.
6. Stabilized municipal government.
7. Increased utility revenues.

The case against—

1. Too many laws already.
2. Danger of political control of licensing board or body.

With the foregoing arguments in mind, it would seem to me that any fair minded judge would declare the Ayes had it.

At the Buffalo Convention, Professor L. V. Carpenter, Chairman of the Committee on Short Schools and Licensing, suggested the following steps in getting licensing adopted:

1. Sell affected employees on its advantages.
2. Pursue vigorous publicity campaigns, stressing the protection to public health.
3. Sell State Health Department on the idea. (This would hardly seem necessary.)
4. Look for a clause in State Public Health Laws which permits State Health Department to issue regulations governing water works operators, etc.

5. If such clause is found, get Attorney General's opinion on legality of setting up suitable licensing regulations under this clause.
6. If laws do not permit this, then prepare a suitable bill for submission to the legislature. The writer would add to this last that in the meantime it might be advantageous to set up a voluntary plan with the idea in mind of increasing the interest in training and raising the standards of water and sewer men while at the same time educating them and officials, as well as the public, to the proposition of licensing.

From the experiences and ideas of many of those interested in Licensing, the following would seem to be good suggestions for those engaged in preparing such a bill.

1. Be brief and do not endeavor to set up detailed regulations.
2. Delegate to the Sanitary Engineering Division of State Health Department or to some competent representative board the authority to set up regulations to govern licensing, determine classifications and qualifications for same, conduct examinations and investigations, issue certificates, handle renewals, etc.
3. Require licenses to be renewed at definite periods, preferably annually.
4. Make license and renewal fees small.
5. Insert "grandfather clause" which will permit a superintendent or operator to continue to operate his same plant or system as long as no important changes are made thereto.
6. A separate tenure bill, affecting only employees of publicly owned plants and systems, should provide for removal of licensed men only after presenting specific charges in writing and holding a public hearing. This bill should also provide that salaries of licensed men may not be reduced—perhaps to force them to leave a position—except by state law or with Health Department approval.

So much for some of the pros and cons of licensing. The Board of Directors of the A.W.W.A., in January 1937, passed the following Resolution: "The Board of Directors of the A.W.W.A., realizing the importance of efficient operation of water purification plants as a protection to public health, endorse licensing, certification, or some form of regulation of water purification plant operators and supervisors as a means of preventing the employment of incompetent

personnel and assuring some continuity of employment of efficient personnel. Sections of the A.W.W.A. are hereby encouraged to engage in voluntary licensing." We have already licensed doctors, nurses, barbers, food handlers, garbage collectors, and many others. Is there any valid reason why we should not license those who provide and handle that commodity which, next to air, is most essential for human life, namely, water?

CORROSION AND THE LANGELIER CALCIUM CARBONATE SATURATION INDEX*

By F. E. DeMartini

(Water Purification Engineer)

(San Francisco Water Department)

It appears desirable to point out, at once, the scope of this paper and the reason for its presentation. At the general convention of this Assocition, held in Los Angeles, in June, 1936, Prof. W. F. Langelier (1) of the University of California, presented one of the most outstanding papers on corrosion control in recent years. His theory and discussion were presented under the title: "The Analytical Control of Anti-Corrosion Water Treatment." A general equation was developed from which the so-called Saturation Index of a water can be determined if the pH, alkalinity, calcium, and total dissolved solids are known. This index indicates whether a given water will tend to deposit or dissolve a calcium carbonate coating. Prof. Langelier pointed out that there were many aspects of the problem to be studied and that future observation would be necessary to establish the practical application of the theory presented.

The Water Purification Division of the California Section (A.W.W.A.) undertook to carry on a number of studies during the past 15 months. One of the studies decided upon was a cooperative investigation, by laboratories of various water departments in the state, on pipe corrosion and the Langelier Saturation Index. Observations were to be made of corrosion conditions in their pipe lines and distribution systems. These findings were to be correlated with the results that might be expected from the application of Langelier's formula to the waters used therein.

The program was started under the direction of Mr. R. F. Goudey who later asked to be excused from the assignment. In May of this year, I was asked to take over the supervision of the program. The laboratories of five California cities have cooperated in this study. They are: Beverly Hills, Oakland and eight other cities

* Presented before the California Section, October, 1937.

served by the East Bay Municipal Utility District, Los Angeles, Pasadena, and San Francisco. This paper may be considered a report of the studies carried out by these cooperating laboratories. The data sent in by these communities have been tabulated, studied, and an attempt made to correlate them. After a preliminary discussion of some of the points investigated, and the work which was done, the data accumulated by the various communities will be presented and finally the conclusions, which seem to be justified, drawn.

Prof. Langelier's general equation is stated as follows:

$$pH_s = (pK_2' - pK_s') + pCa + p\left\{Alk + (H^+) - \frac{K_w}{(H^+)}\right\} + \log\left\{1 + \frac{2K_2'}{(H_s^+)}\right\}$$

Between pH 6.5 and 9.5, the correction to Alk. for H^+ and OH^- is ordinarily small and can be omitted, similarly for the term $1 + \frac{2K_2'}{(H_s^+)}$ between this range. Therefore, the simplified equation is:

$$pH_s = (pK_2' - pK_s') + pCa + pAlk$$

In these expressions, pH_s = pH which the given water should have, to be in equilibrium with calcium carbonate. (Calcium and Alkalinity remaining unchanged.) pK_2' and pK_s' are the negative logarithms of the second dissociation constant for carbonic acid, and the activity product of $CaCO_3$, respectively. The difference $(pK_2' - pK_s')$ varies with ionic strength, salinity and temperature.

pCa is the negative logarithm of the molal concentration of calcium.

pAlk is the negative logarithm of the equivalent concentration of titratable base.

H^+ = molal concentration of H^+ in the water when at actual pH.

H_s^+ = molal concentration of H^+ in the water when it is at pH_s.

By use of the Langelier formula, pH_s is computed. The saturation index is obtained by subtracting pH_s from the actual pH of the water $(pH - pH_s)$. A negative index indicates that the water is undersaturated with calcium carbonate and, hence, tends to dissolve an existing coating at that point. A positive index indicates the reverse, and a zero index indicates a water exactly at equilibrium with calcium carbonate.

It is of interest to note that somewhat later than the presentation of Langelier's paper, that is, in October, 1936, the German investigator, Dr. R. Strohecker (2), working at the University Institute for Food Chemistry, at Frankfort, presented for publication a paper entitled, "A New Method for Determination of the Aggressiveness of Water." By use of the three equations, $\frac{(H^+)(HCO_3^-)}{CO_2} = K_1$, $\frac{(H^+)\ (CO_3^=)}{HCO_3^-} = K_2$, and $(Ca^{++})\ (CO_3^=) = K_L$, he derives a formula for determination of equilibrium pH. The formula is stated as follows: $pH_{equil.} = 11.49 - \log (\text{bound } CO_2) - \log CaO$. By subtracting actual pH from this equilibrium pH, he obtains a figure corresponding to Prof. Langelier's Saturation Index except that it is of opposite algebraic sign. He similarly concludes that waters having a zero index are in equilibrium with calcium carbonate and those deviating from an index of zero tend either to deposit or dissolve calcium carbonate, and, hence, are either aggressive or non-aggressive. It is of importance to know that two individuals in different parts of the world have followed similar reasoning in working out a method for evaluating the aggressive characteristics of a water. In reviewing his work, it was noted that no attempt was made to take into account the ionic strength of the water nor the variation of the constants due to temperature effects. The principal difference between Strohecker's and Langelier's derivations is that the former appears to consider bicarbonate concentration equal to the alkalinity. In waters containing normal carbonate, alkalinity (which allows for the presence of carbonate ions) should be used as well as bicarbonate ion concentration in the development of an equation for general use. Dr. Strohecker's equation can not be used for computing equilibrium pH for waters which have an appreciable carbonate concentration. Most soft waters which have been treated with lime would be of this type, hence, for such waters the equation could not be used to tell whether the waters had attained the equilibrium pH. In the experimental data presented by Dr. Strohecker, raw waters were taken in which the highest actual pH was 7.78, hence, would contain no titratable normal carbonate. For the cases in which this equation holds, Dr. Strohecker's theory and development is similar to that of Langelier's.

It was believed advisable, early in this work, to repeat one of the experiments carried out by Prof. Langelier to verify independently

TABLE 1

Experimental confirmation of the Langelier formula for the saturation pH of natural waters

$$pH_s = (pK'_2 - pK'_s) + pCa + pAlk + \log\left(1 + \frac{2K_2}{(H_2^+)}\right)$$

Waters brought into equilibrium with solid calcium carbonate

SAMPLES FROM	DATE	pH BEFORE SATURATION	RESULTS OF ANALYSIS				$(pK'_2 - pK'_s)$ 25°C.	pCa	pAlk.	$\log\left(\frac{1+2K'_2}{H^+_s}\right)$	pH_s	SATURATION INDEX $pH - pH_s$
			pH	Parts per million								
				Ca^{++}	Total Alkalinity as $CaCO_3$	Total Dissolved Solids						
1. H. H. at Moccasin Res	8/26/37	7.50	9.25	7.7	22	50	2.08	3.72	3.36	.08	9.24	+.01
2. Crystal Springs Res	8/27/37	7.70	8.30	19.9	76	105	2.13	3.30	2.82		8.25	+.05
3. Calaveras Reservoir	8/17/37	8.40	8.05	27.2	98	153	2.17	3.17	2.71		8.05	0.00
4. San Bruno Tap	8/18/37	7.87	7.80	39.6	176	389	2.30	3.00	2.45		7.75	+.05
5. Pleasanton Wells	8/17/37	7.85	7.57	53.6	224	358	2.28	2.87	2.35		7.50	+.07

TABLE 2

Saturation index and CO_2 content of Beverly Hills water

SOURCE OF SAMPLE	DATE	RESULTS ARE IN PPM. (EXCEPT pH VALUES)						pH_s AT 25°C.	pH_s AT 75°C.	SATURATION INDEX pH − pH_s	
		CO_2 Content		Av. pH	Ca	Total Alk. as $CaCO_3$	Total Dis. Solids				
		By titration	By Computation							At 25°C.	At 75°C.
Beverly Hills Influent	Jan.—37	Not Determined	31.3	7.3	61.6	322	678	7.40	7.00	−.10	+.30
Beverly Hills Effluent	Jan.—37		9.7	7.6	26.0	200	590	7.96	7.56	−.36	+.04
Beverly Hills Influent	Feb.—37		24.4	7.4	62.4	317	703	7.42	7.02	−.02	+.38
Beverly Hills Effluent	Feb.—37		9.7	7.6	24.4	200	581	7.98	7.58	−.38	+.02
Beverly Hills Inffluent	Apr.—37		24.8	7.4	60.0	322	638	7.40	7.01	0.00	+.39
Beverly Hills Effluent	Apr.—37		6.3	7.8	24.0	206	615	7.97	7.58	−.17	+.22
Beverly Hills Influent	May—37		23.5	7.4	61.6	305	690	7.42	7.03	−.02	+.37
Beverly Hills Effluent	May—37		8.0	7.7	26.8	207	602	7.92	7.53	−.22	+.17
Beverly Hills Influent	June—37		26.4	7.4	62.4	342	703	7.38	6.98	+.02	+.42
Beverly Hills Effluent	June—37		8.5	7.7	27.6	220	610	7.89	7.50	−.19	+.20
Beverly Hills Inffluent	July—37		33.6	7.3	65.6	346	658	7.34	6.95	−.04	+.35
Beverly Hills Effluent	July—37		11.1	7.6	28.8	229	588	7.85	7.45	−.25	+.15

the validity of the theoretical equation. Therefore, at the San Francisco water department laboratory, five samples of water differing considerably in composition were treated with an excess of calcium carbonate powder. Gallon bottles were filled to the top, tightly stoppered to exclude air, and rotated in a "ball mill" to provide contact and allow the water to come to equilibrium with calcium carbonate. After settling, the clear samples were siphoned off and the pH, total alkalinity, calcium, and total dissolved solids were determined. pH tests were made with a Coleman glass electrode. The data are tabulated (see Table 1) and indicate a maximum deviation of .07 of a pH unit between the observed pH at saturation and the computed pH_s for the treated water. These data thus agree with the similar work done by Prof. Langelier and confirm the validity of the equation.

The principal study carried out by the group of laboratories was the seasonal determination of the saturation index of their supplies. All possible observations of corrosion and rusty water conditions in the pipe lines and systems were also made, as a basis for correlating actual findings with the computed saturation indexes.

RESULTS OF STUDIES

The tabulated results of the determinations are presented for each community in Tables 2 to 11. A brief review of the results follows:

1. *Beverly Hills.* (See Table 2.) The Beverly Hills water is obtained from a large number of deep wells. The supply is softened and filtered. Table 2 is a summary of analyses on samples composited daily over a month's time for influent and effluent of the filter plant. Practically no complaints of rusty or red water and no difficulties reported with corrosion of pipe lines or failures resulting from corrosion. The tabulation shows the water to have a Saturation Index of $+.02$ to $-.38$ at 25°C. and $+.02$ to $+.42$ at 75°C.

2. *East Bay Municipal Utility District.* (See Table 3.) Three principal sources of supply are used by the District. They are Mokelumne, San Pablo, and Upper San Leandro. In 1935–36, the relative quantities supplied by these sources were 11.6, 0.4, 1.3 billion gallons, and in 1936–37, 11.7, 2.7, and 0.5 billion gallons, respectively. The Upper San Leandro and San Pablo supplies are coagulated and filtered. The Mokelumne supply is filtered at Orinda filter plant with or without coagulation, depending upon the season and natural condition of the water. All waters are treated with

TABLE 3

Saturation index of East Bay municipal utility district water supplies

SOURCE OF SAMPLE	PERIOD REPRESENTED	RESULTS ARE IN PPM. pH	Ca	Total Alk. as $CaCO_3$	Total Dis. Solids	pH_s at 25°C.	pH_s at 75°C.	SATURATION INDEX $pH - pH_s$ At 25°C.	At 75°C.	LIME DOSAGE GR./GAL.
San Pablo Reservoir	7/1/35 6/30/36	8.3	21.0	70	165	8.30	7.90	0.00	+.40	.45
San Pablo Reservoir	7/1/36 6/30/37	8.4	23.3	84	175	8.18	7.78	+.22	+.62	.35
Pardee Reservoir	7/1/35 6/30/36	9.0	4.5	15	27	9.07	9.12	−.67	−.12	.26
Pardee Reservoir	7/1/36 6/30/37	9.0	6.2	21	42	9.34	8.86	−.34	+.14	.39
Upper San Leandro Reservoir	7/1/35 6/30/36	8.0	33.3	104	190	7.95	7.55	+.05	+.45	.55
Upper San Leandro Reservoir*	7/1/36 6/30/37	8.2	34.6	118	216	7.90	7.50	+.30	+.70	.35
Chabot Reservoir	7/1/35 6/30/36		39.6	134	250	7.82	7.41			
Chabot Reservoir	7/1/36 6/30/37	8.7	32.9	120	220	7.93	7.53	+.77	+1.17	

* Free CO_2 determined by titration for Upper San Leandro, only, 7/1/36–6/30/37.
By titration = 1.5 ppm.
By computation = 1.4 "

lime to raise the pH for corrosion control. The Mokelumne supply is treated with lime at Pardee Reservoir for protection of the aqueduct and again at Orinda plant, when the water is coagulated, due to the drop in pH resulting from the treatment.

For the San Pablo and Upper San Leandro supplies the Saturation Index is 0 to +.30 in the cold water and +.40 to +.70 in the hot. For Mokelumne water the index in 1935–36 was −.67 in the cold, −.12 in the hot, while in 1936–37 it was −.34 in the cold and +.14 in the hot. During the earlier period when the cold water index was −.67 there were 234 complaints of rusty water, whereas in the latter period when the cold water index was −.34 there were only 32 complaints of rusty water. This supply was treated with lime during both periods but due to a second liming at the Orinda Filter Plant during 1936–37, the calcium and alkalinity were both higher and the pH_s correspondingly lower.

3. *Los Angeles.* (See Tables 4 to 7.) Aqueduct water is obtained from Owens River, and wells in Owens Valley. Other sources are gallery wells in the Los Angeles River and various deep wells throughout the city. Analyses for aqueduct sources and areas served by these sources in 1936 and 1937 show that the Saturation Index varies from −.36 to +.24 in cold water. For river sources and areas served thereby the index varied from −.06 to −.15 in March, 1937. For mixed sources, the index varied from −.32 to +.53 or, if one softened supply is omitted, from −.01 to +.53. It is evident that the aqueduct water is the most likely to cause trouble from the viewpoint of its saturation index.

For corrosion experiences, I quote Mr. R. F. Goudey, Sanitary Engineer, Los Angeles Department of Water & Power: "The small number of corrosion complaints reaching our office indicates that, in general, our water is naturally well conditioned. Most of the corrosion cases which have been brought to our attention have been in the district served with aqueduct water through Stone Canyon Reservoir. The difficulties were always confined to hot water systems. A few isolated cases of corrosion in hot water systems have also been reported from the area served with Los Angeles River water. These cases, however, have been limited to places where the water had been softened by the zeolite process. A review of the data on hand indicates that water with a saturation index more positive than −.45 has not caused appreciable corrosion in cold water systems. Water with an index between zero and −0.3 has caused corrosion in hot water systems."

TABLE 4

Saturation index and CO_2 content of Los Angeles water supply sources

SAMPLING POINT	DATE	TEMP., °C.	pH	CO_2 CONTENT, PPM.		SATURATION INDEX pH − pH_s
				Determined by S.M.W.A.* Method	Computed	
Aqueduct Sources						
Low. San Fernando Outlet	9/18/36	24	8.2	0	1.4	+0.03
" " "	12/18/36	12	8.2	0	1.5	−0.11
" " "	3/19/37	13	8.2	0	1.6	+0.05
" " "	6/18/37	21	8.2	0	2.1	+0.26
Lower Hollywood	9/ 6/36	21	8.1	0	2.3	+0.23
" "	12/ 2/36	15	7.9	1.0	2.8	−0.37
" "	3/ 3/37	13	8.3	0	1.2	+0.19
" "	6/ 9/37	17	8.2	0	1.7	+0.16
Lower Franklin	9/ 1/36	24	7.9	2.6	2.9	−0.27
" "	12/ 3/36	15	8.0	0	2.3	−0.32
" "	3/ 4/37	13	8.0	0.5	2.6	−0.08
" "	6/ 1/37	18	8.3	0	1.4	+0.28
Stone Canyon	9/10/36	21	8.2	0	1.7	+0.09
" "	12/10/36	16	7.9	1.0	3.0	−0.36
" "	3/11/37	13	7.9	0.5	3.0	−0.35
" "	6/ 8/37	19	8.1	0	2.0	+0.03
River Sources						
Riverside Pumps	3/26/37	18	7.65	6.5	8.0	−0.15
Buena Vista Res	3/26/37	17	7.78	3.2	5.8	−0.06
Mixed Sources						
Silver Lake Res	9/ 9/36	24	7.9	0	3.7	+0.16
" " "	12/ 9/36	16	8.4+	0	1.1	+0.53
" " "	6/ 9/37	21	8.3	0	1.6	+0.65
Elysian Res	3/26/37	18	7.78	2.3	5.4	+0.01

* S.M.W.A.—Standard Methods of Water Analysis.

TABLE 5

Saturation index and CO_2 content of Los Angeles water supply distribution system

SAMPLING POINT	DATE	TEMP., °C.	pH	CO_2 CONTENT, PPM.		SATURATION INDEX $pH - pH_s$
				Determined by S.M.W.A.* Method	Computed	
Aqueduct Water						
Pico & Overland	4/16/37	17	8.15	0	1.6	−0.02
Westwood & Mississippi	"	17	8.11	0	1.8	−0.04
Westwood & Wilshire	"	17	8.12	0	1.8	−0.04
605 Funchal Road—Cold	1/13/37	14	8.32	0	1.1	+0.09
605 Funchal Road—Hot	"	60	8.07	1.9	1.8	+0.64
Bel-Air Country Club—Cold	"	16	8.20	0	1.4	−0.02
Bel-Air Country Club—Hot	"	41	8.01	2.2	2.2	+0.24
River Water						
Avenue 55	3/26/37	16	7.58	5.2	9.3	−0.14
Avenue 28	"	16	7.59	4.4	9.1	−0.13
Mixed Water						
5th & Olive—Raw Water	May '37	17	7.78	6.5	5.7	+0.07
5th & Olive—Softened Water	"	18	8.29	0	1.7	−0.32
5th & Olive—Hot Boiler Water	"	100	9.20	0	0	+2.07
5th & Olive—Cold " "	"	25	11.14	0	0	+2.74

* S.M.W.A.—Standard Methods of Water Analysis.

* S.M.W.A.—Standard Methods of Water Analysis.

TABLE 6

Saturation index and CO_2 content, Owens Valley aqueduct

Data by courtesy of *J. E. Phillips*, Engineer, Los Angeles Aqueduct, Dept. of Water & Power, Los Angeles, California

SAMPLING POINT	DATE	TEMP., °C.	pH	CO_2 CONTENT, PPM.		SATURATION INDEX $pH - pH_s$
				Determined by S.M.W.A.* Method	Computed	
Haiwee Res. Outlet	8/12/36	23	8.24	0	0	−0.09
600′ Below Power Plant	"	23	8.30	0	0	+0.03
Little Lake Station	"	23	8.26	0	0	−0.12
Nine Mile Syphon	"	23	7.65	3.5	3.9	−0.61
Freeman Station	"	23	7.80	4.0	2.8	−0.45
Jawbone Syphon	"	24	7.83	3.5	2.6	−0.41
Antelope Syphon	"	24	7.90	4.0	2.2	−0.32
Fairmont Res. Inlet	8/13/36	23	7.75	5.0	3.2	−0.48
Fairmont Res. Outlet	"	23	7.98	3.0	1.9	−0.27
No. 1 Power House Inlet	"	24	8.08	1.5	1.5	−0.16
Dry Canyon Inlet	"	24	8.10	1.0	1.5	−0.05
Upper San Fernando Inlet	"	24	8.00	1.0	2.0	−0.18

* S.M.W.A.—Standard Methods of Water Analysis.

TABLE 6A

Saturation index and CO_2 content, Owens Valley aqueduct

Data by courtesy of *J. E. Phillips*, Engineer, Los Angeles Aqueduct, Dept. of Water & Power, Los Angeles, California

SAMPLING POINT	DATE	TEMP., °C.	pH	CO_2 CONTENT, PPM.		SATURATION INDEX $pH - pH_s$
				Determined by S.M.W.A.* Method	Computed	
Haiwee Res. Outlet	10/5/37	17.0	8.65	0	0	+0.39
600′ Below Power House	"	17.5	8.70	0	0	+0.45
Little Lake Station	"	18.0	8.70	0	0	+0.46
Nine Mile Syphon North	"	18.0	8.69	0	0	+0.45
Nine Mile Syphon South	"	18.0	8.70	0	0	+0.46
Freeman Station	"	18.5	8.71	0	0	+0.48
Jawbone Syphon South	"	19.0	8.71	0	0	+0.49
Antelope Syphon South	"	19.0	8.61	0	0	+0.41

* S.M.W.A.—Standard Methods of Water Analysis.

TABLE 7

Saturation index and CO_2 content of Owens Valley aqueduct, 24 hour survey

TIME OF SAMPLING	DATE	SAMPLES TAKEN AT LITTLE LAKE STATION					SAMPLES TAKEN AT SODA HILL STATION				
		Temp., °C.	pH	Free Carbon Dioxide		Saturation Index pH − pH_s	Temp., °C.	pH	Free Carbon Dioxide		Saturation Index pH − pH_s
				S.M.W.A.	Computed				S.M.W.A.	Computed	
6:00 A.M.	3/3/37	10	8.21	0	1.6	−0.09					
7:00 A.M.	"	10	8.36	0	1.1	+0.07					
8:00 A.M.	"	10	8.32	0	1.2	+0.01	10	7.93	1.6	3.0	−0.37
9:00 A.M.	"	10	8.24	0	1.5	−0.05	10	7.87	1.6	3.4	−0.42
10:00 A.M.	"	10	8.26	0	1.4	−0.04	10	7.94	2.7	2.9	−0.35
11:00 A.M.	"	10	8.28	0	1.3	−0.01	10	8.03	2.3	2.4	−0.25
12:00 Noon	"	10	8.30	0	1.3	+0.01	10	7.86	1.6	3.5	−0.44
1:00 P.M.	"	10	8.27	0	1.4	−0.02	10	7.84	2.2	3.6	−0.46
2:00 P.M.	"	10	8.13	0	1.9	−0.17	10	7.85	2.7	3.6	−0.45
3:00 P.M.	"	10	8.27	0	1.4	−0.03	10	7.86	2.7	3.5	−0.44
4:00 P.M.	"	10	8.22	0	1.5	−0.08	10	7.87	2.2	3.4	−0.43
5:00 P.M.	"	10	8.20	0	1.6	−0.10	10	7.89	1.6	3.2	−0.41
6:00 P.M.	"	10	8.32	0	1.2	+0.04	10	8.04	1.6	2.3	−0.26
7:00 P.M.	"	10	8.30	0	1.0	−0.02	10	8.01	1.6	2.5	−0.27
8:00 P.M.	"	10	8.31	0	1.2	+0.01	10	8.07	0	2.1	−0.22
9:00 P.M.	"	10	8.33	0	1.2	+0.04	10	8.01	0.5	2.5	−0.28
10:00 P.M.	"	10	8.26	0	1.4	−0.03	10	8.13	0	1.9	−0.15
11:00 P.M.	"	10	8.24	0	1.4	−0.06	10	8.10	0	2.0	−0.18
12:00 Midnight	"	10	8.32	0	1.2	+0.02	10	7.86	0.5	3.5	−0.43
1:00 A.M.	3/4/37	10	8.32	0	1.2	+0.03	10	7.89	0.5	3.2	−0.40
2:00 A.M.	"	10	8.21	0	1.6	−0.08	10	7.92	0.5	3.0	−0.37
3:00 A.M.	"	10	8.33	0	1.2	+0.04	10	7.92	0.5	3.0	−0.37
4:00 A.M.	"	10	8.21	0	1.6	−0.08	10	7.84	1.6	3.6	−0.45
5:00 A.M.	"	10	8.28	0	1.3	−0.03	10	7.92	1.6	3.1	−0.35
6:00 A.M.	"	10	8.29	0	1.3	0	10	8.18	0	1.7	−0.10
7:00 A.M.	"						10	8.22	0	1.5	−0.06

S.M.W.A.—Standard Methods of Water Analysis. (Titration with N/44 NaOH and phenolphthalein indicator.)

Data in above table by courtesy of J. E. Phillips, Engineer, Los Angeles Aqueduct, Department of Water & Power, Los Angeles, California.

Saturation index values are available for the Los Angeles Aqueduct along its length. The index varies from −.61 to +.03. It is noteworthy to observe from Mr. Goudey's notes that, except for corrosion effects in Nine Mile Siphon, unprotected steel surfaces have shown no ill effects. At Nine Mile Siphon, the index was −.61. The next lowest index in the series of tests made along the aqueduct was −.45. Table 7 is interesting in that the hourly fluctuations in saturation index and free carbon dioxide for 24 hours are shown for stations above and below the tunnel sections into which carbon dioxide enters. Tables 6 and 6A give the free carbon dioxide content and Saturation Index for points along the aqueduct, before and after controlling entry of carbon dioxide into the Soda Hill Tunnel. Free carbon dioxide has been eliminated and the Saturation Index shifted from around −.1 to −.5 up to +.4 to +.5.

4. *Pasadena.* (See Table 8.) Pasadena obtains its supply from a large number of deep wells, several infiltration tunnels, and a surface supply impounded in Morris Reservoir. In water at 25°C., the saturation index varied from −.63 to +.32 and at 75°C., from −.23 to +.72. If we eliminate one well supply, the index varies from −.05 to +.72. In commenting on the corrosion difficulties experienced, Mr. C. W. Sopp, Ass't. Chief Engineer of the Pasadena Water Department stated: "Inspection of the table reveals that most of the waters have a small corrosive tendency at temperatures of 25°C., while at 75°C., or the approximate temperature of hot water systems, they have a positive saturation index showing a tendency to deposit calcium carbonate. This is verified by deposits found in hot water heaters. We have little definite information on the relative corrosion occurring in the mains from the different sources of supply. In general, it appears that the corrosion in all parts of the system, is about what normally occurs, being general tuberculation, but not in excessive amounts. We recently had a survey made of our transmission mains by the Pitometer Company. The values of C found by Williams & Hazen's formula closely agreed with that to be expected for a similar class and age of mains."

Regarding one source, the Devils Gate Tunnel, Mr. Sopp says: "You will note that the dissolved carbon dioxide in the Devils Gate Tunnel water is very high, whereas the computed saturation index at 77°F. is always above −1.0, which would indicate that the water is not actively corrosive, according to Langelier's formula. However, we first began to have trouble with this water in February, 1935.

TABLE 8

Saturation index and CO_2 content of Pasadena water supply

SOURCE OF SAMPLE	DATE	RESULTS ARE IN PPM. CO_2 Content		pH	Ca	Total Alk. as $CaCO_3$	Total Dis. Solids	pH_s AT 25°C.	pH_s AT 75°C.	SATURATION INDEX pH − pH_s	
		By Titration	By Computation							At 25°C.	At 75°C.
Arroyo Well	1935-36	4.0	3.5	7.95	38.1	164	312	7.76	7.36	+0.19	+.59
Copelin Well	1935-36	0.0	6.0	7.71	37.0	158	305	7.80	7.40	−0.09	+.31
Devil's Gate Motor	1935-36	13.0	4.1	7.74	30.6	115	232	7.98	7.58	−0.24	+.16
Eaton Canyon	1935-36	1.0	3.1	7.95	37.0	142	266	7.83	7.43	+0.12	+.52
Garfield Well	1935-36	1.5	3.3	7.75	22.9	96	212	8.18	7.78	−0.43	−.03
Millard Canyon	1935-36	3.0	3.4	7.95	31.4	154	304	7.88	7.48	+0.07	+.47
Morris Reservoir	1935-36	0.0	3.5	7.94	46.3	160	296	7.70	7.30	+0.24	+.64
Morris Reservoir	5-17-37	0.0	5.5	7.70	42.3	142	211	7.74	7.34	−0.04	+.36
Post Tunnel	1935-36	18.0	8.3	7.62	43.5	180	343	7.68	7.28	−0.06	+.34
Post Well	1935-36	23.5	7.6	7.62	46.6	162	338	7.71	7.31	−0.09	+.31
Sunset Well	1935-36	3.5	3.7	7.87	32.0	142	278	7.90	7.50	−0.03	+.37
Ventura Well	1935-36	9.8	7.4	7.58	38.0	146	274	7.81	7.41	−0.23	+.17
Villa Well	1935-36	3.0	4.4	7.60	22.2	90	227	8.23	7.83	−0.63	−.23
Windsor Well	1935-36	9.0	8.3	7.62	40.1	178	326	7.72	7.32	−0.10	+.30
Woodbury Well	1935-36	0.0	3.4	7.78	20.2	106	229	8.19	7.79	−0.41	−.01
Figueroa Well	1935-36	49.0	36.0	7.23	85.4	311	551	7.23	6.83	0.0	+.40
Devils Gate Tunnel	1935-36	45.0	26.0	7.21	65.1	227	405	7.43	7.03	−0.22	+.18
Devils Gate Tunnel	3-17-37	12.0	25.0	7.16	50.9	182	329	7.61	7.21	−0.45	−.05
Devils Gate Tunnel	5 -6-36	37.0	40.0	7.20	90.7	322	552	7.19	6.79	+0.01	+.41
Devils Gate Tunnel	3-11-36	52.0	28.0	7.38	94.1	337	573	7.17	6.77	+0.21	+.61
Devils Gate Tunnel	7-30-35	47.5	12.0	7.65	75.4	269	490	7.33	6.93	+0.32	+.72

TABLE 9

Saturation index and CO_2 content of San Francisco water Crystal Springs Reservoir and distribution system samples

SOURCE	DATE	pH	RESULTS IN PPM. Free CO_2 By Titration	Free CO_2 By Computation	Ca	Total Alk. as $CaCO_3$	Total Dis. Solids	pH_s AT 25°C.	pH_s AT 75°C.	$CaCO_3$ SATURATION INDEX $pH - pH_s$ 25°C.	75°C.
Crystal Springs Res.	5/ 1/34	7.7		4.3	41	113	226	7.86	7.47	−.16	+.23
" " "	10/17/34	8.0		2.6	46	134	252	7.74	7.34	+.26	+.66
" " Pumps	2/ 8/35	7.7		3.2	22	84	201	8.23	7.83	−.53	−.13
" " Line	7/17/35	7.3		6.2	17	64	118	8.41	8.01	−1.11	−.71
" " Res.	1/ 9/36	7.3	1.0	6.0	17	62	102	8.41	8.01	−1.11	−.71
" " Pumps	2/24/36	7.5	1.5	4.7	18	76	142	8.33	7.94	−.83	−.44
" " Res.	5/20/36	7.7	1.0	2.3	14	60	166	8.55	8.16	−.85	−.46
" " "	10/19/36	7.6	1.0	2.1	10	44	45	8.72	8.33	−1.12	−.73
" " "	2/10/37	7.5	1.0	2.8	11	46	34	8.65	8.25	−1.15	−.75
" " Line	5/11/37	7.3	1.5	4.9	13	50	91	8.61	8.20	−1.31	−.90
" " "	8/23/37	7.8		2.2	18	71	130	8.35	7.95	−.55	−.15
Distribution Systems											
University Mound System	5/11/37	7.3		8.7	12.9	50	90	8.61	8.20	−1.31	−.90
College Hill System	5/11/37	7.7		3.9	14.2	55	100	8.54	8.13	−.84	−.43
Lake Honda Reservoir	5/11/37	7.5		6.1	14.4	56	100	8.52	8.11	−1.02	−.61
University Mound Outlet	8/23/37	7.73	1.0	2.3	16.0	65	120	8.44	8.04	−.71	−.31
University Mound System	8/23/37	7.79	1.5	2.3	16.0	72	120	8.39	7.99	−.60	−.20
College Hill Outlet	8/23/37	7.71	1.5	2.4	15.0	64	124	8.46	8.06	−.75	−.35
College Hill System	8/23/37	7.75	1.0	2.3	15.0	67	124	8.44	8.04	−.69	−.29
Lake Honda Outlet	8/23/37	7.70	1.5	2.5	15.3	65	125	8.46	8.06	−.76	−.36
Lake Honda System	8/23/37	8.00	0	1.2	15.3	60	125	8.49	8.05	−.49	−.05

Lake Honda System	8/23/37	8.00	0	1.2	15.3	60	125	8.49	8.05	−.49	−.05

TABLE 10

Saturation index and CO_2 content of San Francisco water—Calaveras Reservoir Samples

SOURCE	DATE	pH	RESULTS IN P.P.M.					pH_s AT 25°C.	pH_s AT 75°C.	$CaCO_3$ SATURATION INDEX (pH − pH_s)	
			Free CO_2		Ca	Total Alk. as $CaCO_3$	Total Dis. Solids				
			Titr.	Comp.						25°C.	75°C.
Calaveras Reservoir	8/ 7/34	7.7		4.1	47	106	195	7.80	7.40	−.10	+.30
" "	4/22/35	7.8		3.2	19	103	174	8.19	7.79	−.39	+.01
" "	3/ 5/36	8.3		0	20	74	149	8.30	7.90	0	+.40
" "	4/13/36	7.6		3.6	23	74	146	8.24	7.84	−.64	−.24
" "	10/19/36	8.3		0	22	88	138	8.17	7.78	+.13	+.52
" "	2/10/37	7.9		2.1	26	88	108	8.07	7.68	−.17	+.22
" "	5/11/37	7.7		3.0	23	78	123	8.20	7.81	−.50	−.11
" "	8/17/37	8.5		0	28	94	156	8.06	7.66	+.44	+.84

Note: High pH in reservoir waters due to surface samples and heavy algae growths.

TABLE 11

Hetch Hetchy System

Pulgas Weir	8/27/36	6.6		8.3	4.0	17	45	9.7	9.2	−3.1	−2.6
Alameda East Portal	8/27/36	6.7		6.2	3.7	16	36	9.8	9.3	−3.1	−2.6
Irvington Portal	8/27/36	6.7		7.0	4.0	18	40	9.6	9.2	−2.9	−2.5
Moccasin Reservoir	4/22/37	7.4		1.0	3.0	13	22	9.9	9.4	−2.5	−2.0
Hetch Hetchy Reservoir	4/26/37	7.0		1.5	2.2	8	16	10.4	9.8	−3.4	−2.8
Red Mountain Bar	5/20/37	6.7	1.0	3.9	1.8	10	20	10.5	9.9	−3.8	−3.2
Oakdale Portal	5/21/37	7.0	1.0	4.3	9.0	22	50	9.2	8.7	−2.2	−1.7
Moccasin Reservoir	9/21/37	6.7	2.0	2.3	1.5	6	17	10.5+	10.3	−3.8+	−3.6
Alameda East Portal	9/21/37	7.9	0	0	7.6	22	41	9.2	8.8	−1.3	−.9

The water had a sharp metallic taste and gave us red water troubles. It is believed that in this case the computed saturation index was not a true indication of corrosion."

5. *San Francisco.* (See Tables 9, 10, 11.) The water served in San Francisco may be divided generally into two supplies: Peninsula Water, which is a mixture of local surface run-off with Hetch Hetchy water from the Sierra Nevada Mountains and Hetch Hetchy Water only, served directly to some consumers on the peninsula. The former supply, before advent of Hetch Hetchy, was used in San Francisco together with Calaveras water. At that time the index for the peninsula lakes was very close to 0, varying from +.26 to −.16 in the analyses available. Calaveras still has an index varying from −.65 to +.44 depending upon the pH in the lake and level from which water is drawn. Crystal Springs Reservoir, the largest peninsula storage reservoir and representative of the water now served in San Francisco, has changed from an index of −.26 just before Hetch Hetchy water was discharged into it, to −1.31.

Hot water index has changed from +.66 to −.70. Corrosion complaints and complaints of rusty water have increased along with the drop in the saturation index. Complaints of failures of hot water pipes, leaks in boiler tubes, and boilers, are many times greater than when the former supply was used. During the past year, some two dozen complaints of serious corrosion troubles were investigated by the purification division. Only cases where complaints were serious due to leaks in house piping, boilers, etc., were investigated.

With regard to Hetch Hetchy water, the cold water index varies from −2.2 to −3.8, and at 75°C. it is −1.7 to −3.2. During the time that this supply was furnished to some of the Peninsula consumers, complaints of rusty and red water were numerous. Red stained sinks and bath tubs were the rule. Inspections of the San Joaquin Valley pipe line indicated that at points where the protective dip was gone, corrosion had taken place readily. Pits up to $\frac{3}{64}''$ (.047″) in depth could be measured after two year's service of this line. At Moccasin Power House, where this water is used, hot water pipes have failed after only a few months of use, and copper piping is now used in all new construction. Piping on hot water heaters at Moccasin Power House has frequently been replaced after only six month's use.

Mention should be made of steps that are being taken to remedy

the situation. Lime treatment of the Peninsula Supply is contemplated to overcome difficulties in San Francisco. To protect the San Joaquin Valley pipe line, treatment of the Hetch Hetchy Supply is planned. At present, an experimental plant is being operated to determine the best treatment, dosages, etc., to use. Application of calcium carbonate powder together with lime has been tried. Due to the low initial alkalinity (about 5 ppm.) it was believed that it would be necessary to supply more alkalinity to the water which could be converted to carbonate ion upon increasing the pH. If calcium carbonate could be dissolved directly, both calcium and carbonate ions could be added at once, in a cheap form, and pH, thereby reduced. The results have indicated that we can attain a pH of 8.8 by means of calcium carbonate only and that lime and calcium carbonate together will probably be the proper treatment for establishing and maintaining the equilibrium pH or a value close to it in the 150 mile aqueduct.

In the experimental plant mentioned, the calcium carbonate powder used met the following specifications: It contains at least 98 percent calcium carbonate; 99.5 percent passes a 325 mesh sieve; the surface area is at least 3,000 sq. cm. per gram. These specifications were based upon results of laboratory experiments with limestone ground to various degrees of fineness. The rate of solution of a substance is a function of surface area and experiments showed that material of the fineness stated gave greater solubility of calcium carbonate and therefore allowed such treatment to be economically possible.

6. *San Mateo.* Table No. 12 is a compilation of results on raw, and lime-treated, Crystal Springs water used in San Mateo by the California Water Service Company. These results show what can be expected as a result of the proposed lime treatment of the San Francisco supply. The saturation index at San Mateo has been shifted to a definite positive value and corrosion difficulties have been greatly reduced. It may not be necessary to treat so heavily, that is, to attain such a high positive index, except in the early stages of lime treatment.

ASSOCIATED INVESTIGATIONS

During the course of this investigation, other associated problems were studied. One was the comparison of free carbon dioxide content, as determined by titration, (with N/44 Sodium Hydroxide in

TABLE 12

Saturation index studies of raw and lime-treated Crystal Springs water made at San Mateo, California through coöperation of Calif. Water Service Co.

SAMPLE	DATE	Ca	ALK.	TOTAL SOLIDS	pH	pH_s at 25°C.	SAT. INDEX	Fe	D.O., % SAT.	REMARKS
Samples from Station No. 6										
C. S. Water at Inlet	6/15/37	13.7	56	100	7.1	8.55	−1.5		58.0	Water not treated with lime, red water complaints numerous
Mixed Water at Reservoir	"	14.1	58	160	7.3	8.56	−1.3	.19	82.5	
Near Reservoir	"	14.1	58	160	7.5	8.56	−1.1	.28	68.5	
End of System	"	14.1	58	160	7.4	8.56	−1.2	.28	33.0	
C. S. Water at Inlet	8/25/37	18.6	69	125	7.6	8.34	−.7	.18	83.3	Water treated with lime. Complaints of red water greatly reduced
Mixed Water and Lime	"	23.4	74	170	8.9	8.24	+.7	.10	88.8	
Near Reservoir	"	23.4	75	170	8.9	8.23	+.7	.08	80.3	
Near End of System	"	23.4	75	170	8.8	8.23	+.6	.12	62.6	
Samples from Station No. 7										
C. S. Water at Inlet	6/17/37	13.7	60	100	7.1	8.52	−1.4			Water treated with lime for corrosion control. Red water complaints practically eliminated
C. S. Water & Lime at Res	"	17.7	66	110	8.6	8.37	+.2	.12		
Near Reservoir	"	17.7	66	110	8.6	8.37	+.2	.09		
End of System	"	17.7	66	110	8.5	8.37	+.1			
C. S. Water at Inlet	8/25/37	18.6	69	122	7.6	8.34	−.7	.18	83.3	Lime treatment in use. Complaints negligible
C. S. Water & Lime at Res	"	21.7	79	130	8.8	8.22	+.6	.08	93.4	
Near Reservoir	"	21.7	79	130	8.8	8.22	+.6	.08	88.2	
End of System	"	21.7	79	130	8.7	8.22	+.5	.24	72.7	

accordance with the procedure in Standard Methods) with the value obtained by computation from the pH and alkalinity of the water. Work by Tillmans (3) depended upon the determination of free carbon dioxide to determine the "aggressiveness" of water. Greenfield and Baker (4) determined free carbon dioxide in order to compute the pH of a given water. Langelier points out that the free carbon dioxide determination is not accurate as ordinarily performed, when only a few p.p.m. of carbon dioxide are present, and particularly so in the neighborhood of pH 8.0 to 8.3 where we are especially interested in its value. To check this point, carbon dioxide titrations were made by the various laboratories and the free carbon dioxide was also calculated from the mass equation for dissociation of carbonic acid, i.e.:

$$\frac{(H^+)(HCO_3^-)}{CO_2} = K_1 = 4.54 \times 10^{-7}$$

$$(H^+) = \frac{CO_2}{(HCO_3^-)} \times K_1$$

$$\text{pH} = \log\left\{\frac{(HCO_3^-) \text{ as } CaCO_3 \times .88}{CO_2 \text{ ppm.} \times 4.54 \times 10^{-7}}\right\}$$

$$= \log\left\{.1938 \times 10^7 \frac{HCO_3^- \text{ as } CaCO_3}{\text{Free } CO_2}\right\} \quad (1)$$

$$\text{pH} = \log (HCO_3^- \text{ as } CaCO_3) + \log (.1938 \times 10^7) - \log CO_2$$

$$\text{or} \qquad \log CO_2 = 6.2874 + \log (HCO_3^- \text{ as } CaCO_3) - \text{pH} \quad (2)$$

Note. In eq. (1) and (2) HCO_3^- and CO_2 are expressed in ppm. It is to be noted that in Tillmans' equation, generally stated as,

$$\text{pH} = \log\left\{.203 \times 10^7 \times \frac{\text{Alk in ppm. } CaCO_3}{\text{Free } CO_2 \text{ in ppm.}}\right\}$$

the factor .61 was used in converting alkalinity as calcium carbonate to its equivalent of carbon dioxide instead of .88, and the value 3.07×10^{-7} was used for K_1 instead of 4.54×10^{-7}. These substitutions have opposite effects so that equation (1) above is only slightly different from Tillmans'. The value of $K_1 = 4.54 \times 10^{-7}$ is taken from the data published by MacInnes and Belcher in 1933. (5) Their value is the result of a very excellent study.

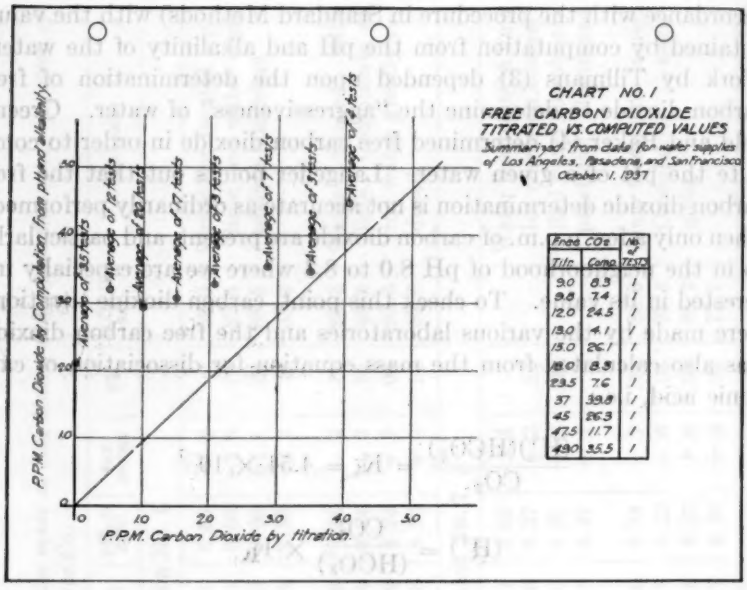

CHART 1

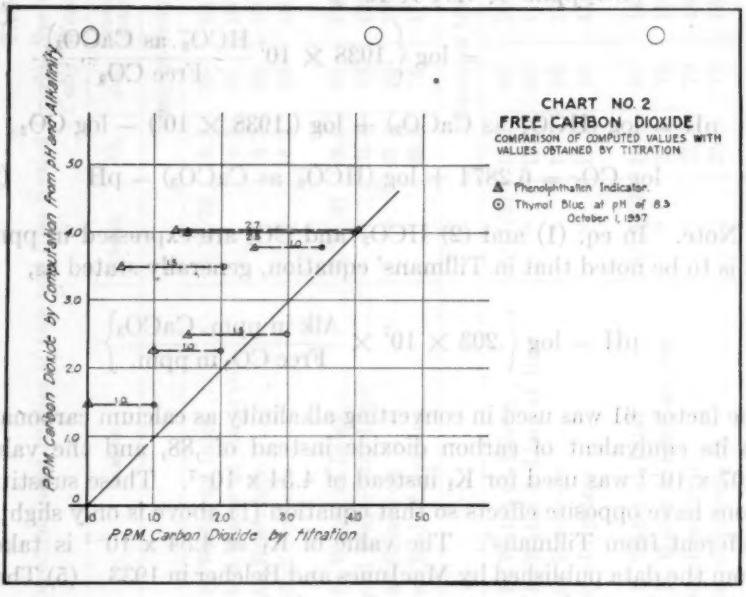

CHART 2

In the various tables shown, the value given as computed carbon dioxide was obtained in all cases by use of equation (2).

The results of the free carbon dioxide tests are shown on Chart 1. The values for titrated carbon dioxide from all sources were grouped so that all those giving the same carbon dioxide value are in one group. For each determination in this group the *computed* carbon dioxide was obtained and the average of these *computed* carbon dioxide values was found. Chart 1 shows this information graphically. It is evident that at low free carbon dioxide contents, almost any titrated value can be obtained, depending upon technique, personal equation, and other factors. We believe this data substantiates the contention that free carbon dioxide values, as ordinarily determined by titration, are inaccurate when the free carbon dioxide content is low.

In the San Francisco laboratory, we noted that phenolphthalein changed color at pH values as low as 7.9, particularly in waters of high alkalinity, instead of at 8.3, the theoretical pH reading at which all free carbon dioxide has been converted to bicarbonate. A number of tests were made to determine if titration to pH 8.3 would give closer agreement with the calculated carbon dioxide. Method followed was to add N/44 sodium hydroxide, in increasing amounts, to a series of 100 cc. samples of a given water. The pH values were then obtained by colorimetric and glass electrode methods. The free carbon dioxide content present with titration to pH 8.3 is obtained from the quantity of N/44 sodium hydroxide required to give a pH of 8.3. Chart 2 indicates the difference in titrated carbon dioxide, obtained by this method, and with phenolphthalein. We believe this factor to be one of those which cause error in titrated carbon dioxide. An indicator with definite color change at pH 8.3 would be the ideal answer to this problem.

Another problem is closely related to this weakness in the determination of free carbon dioxide by titration and is based on the same objections. The method for determining the concentration of normal carbonate, as given in Standard Methods, by means of a two indicator titration is (within a certain range of pH) insufficiently precise for some purposes. Langelier proposed equations for computing bicarbonate and carbonate content and suggested that similar formulae for carbonic acid and hydroxyl ion be used. I believe his proposal is sound and repeat here these formulae with those for carbonic acid and hydroxyl ion added thereto:

$$(1) \qquad (HCO_3^-) = \frac{Alk}{1 + \frac{2K_2'}{(H^+)}}$$

$$(2) \qquad (CO_3^=) = \frac{K_2'}{(H^+)} \times \frac{Alk}{1 + \frac{2K_2'}{(H^+)}}$$

$$(3) \qquad H_2CO_3 = \frac{(H^+)(HCO_3^-)}{K_1} = \frac{(H^+)}{K_1} \times \frac{Alk}{1 + \frac{2K_2'}{(H^+)}}$$

$$(4) \qquad OH^- = \frac{10^{-14}}{(H^+)}$$

Note: (HCO_3^-), $(CO_3^=)$, (H_2CO_3), (OH^-), and (H^+), in these expressions are in mols per liter. Alk. is expressed in equivalents per liter.

For general use in waters not limited to the pH range (4.5 to 10.3) the alkalinity terms should be corrected giving the equations in the form (1a), (2a), and (3a), equation 4 remaining unchanged.

$$(1a) \qquad HCO_3^- = \frac{\left\{Alk + (H^+) - \frac{K_w}{(H^+)}\right\}}{1 + \frac{2K_2'}{(H^+)}}$$

$$(2a) \qquad CO_3^= = \frac{K_2'}{(H^+)} \times \frac{\left\{Alk + (H^+) - \frac{K_w}{(H^+)}\right\}}{1 + \frac{2K_2'}{(H^+)}}$$

$$(3a) \qquad H_2CO_3 = \frac{(H^+)}{K_1} \times \frac{\left\{Alk + (H^+) - \frac{K_w}{(H^+)}\right\}}{1 + \frac{2K_2'}{(H^+)}}$$

From eq. (4) we find that at pH values of 9.0, 10.0, 11.0, and 12.0, the OH^- concentration of a solution expressed as ppm. of $CaCO_3$ will be .5, 5.0, 50, and 500 ppm. respectively, or that the titratable caustic alkalinity is only appreciable at pH values above 10.0 and becomes considerable between pH 11.0 and 12.0.

SUMMARY AND CONCLUSIONS

The theory and equations presented by Prof. W. F. Langelier for calculation of the pH at which a given water is in equilibrium with solid calcium carbonate are briefly reviewed. Similar work by the German investigator, Dr. R. Strohecker, is mentioned and compared with the former work. For five California cities, Beverly Hills, Oakland, Pasadena, Los Angeles, and San Francisco, analytical results and computations of the calcium carbonate saturation index are given in tabulated form. The observations of actual corrosion conditions which have been made in each community are reported. Experiences with corrosion and plans or treatment to overcome the difficulties are mentioned. Some weaknesses in the method of determining free carbon dioxide by titration with N/44 sodium hydroxide and phenolphthalein as the indicator are pointed out. Equations for computing bicarbonate ion, carbonate ion, carbonic acid (or free carbon dioxide) and hydroxyl ion from the total alkalinity and the hydrogen ion concentration are presented.

On the basis of the data accumulated through the coöperation of the five California cities mentioned above, the following conclusions are drawn.

Practical observations of field conditions, summarized here, confirm the theory and formula of Langelier, within the limits indicated:–

(a) *Beverly Hills.* Corrosion difficulties slight or absent. Saturation index varies from +.02 to −.38.

(b) *Oakland or East Bay Municipal Utility District.* Corrosion difficulties slight. Saturation index −.67 in 1935–36 and −.34 in 1936–37 for Mokelumne water. In the former period, corrosion or red water complaints 234, in the latter period, complaints 32.

(c) *Los Angeles.* "Where saturation index has been more positive than −.45 in cold water or above zero in hot water systems no appreciable corrosion has occurred." If water with lower values was used, trouble was experienced.

(d) *Pasadena.* Saturation index in cold water varies from −.63 to +.32. Corrosion in the system reported to be normal tuberculation, but not classed as excessive or troublesome. In one case, however, Devils Gate Tunnel water, a sharp metallic taste and red water troubles were noticed, whereas

the saturation index is between +.32 to −.45. This is not in accordance with the theory. Free carbon dioxide content of this water is 12 to 40 ppm.

(e) *San Francisco.* Local waters before 1934, having an index of +.26 to −.16, gave no particular corrosion problems. Local waters now, having an index of −1.31, are decidedly more corrosive as indicated by a large increase in number of complaints of pipe failures and rusty water. Hetch Hetchy water with an index of −2.2 to −3.8 has caused an excessive number of red water complaints where used directly, and very rapid corrosion and pitting of transmission lines and house piping as at Moccasin Power House.

Hence, in the case of cold waters that have a saturation index more positive than −.5, little corrosion difficulty is to be expected. For hot waters having an index more positive than zero, slight difficulty is to be expected.

This should not be interpreted to mean that waters meeting this standard will be absolutely free from corrosion effects. Pitting may continue, but at a much slower rate. It may also require considerable time for waters brought to the above standard to build up a protective coating on the metal.

The data presented certainly indicate the practical value of the theory and equation of Prof. Langelier. The saturation index obtained shows whether a given water tends to dissolve or deposit a calcium carbonate coating. The equation is not a remedy for the corrosion problem. It is a useful tool for determining, in a direct manner, the aggressive or non-aggressive characteristics of a water. The calculation of the saturation index is simple even though the theory and derivation of the equation may seem complicated.

Incidental to the main problem, the work done indicates weaknesses in the titration method for determination of small quantities of free carbon dioxide.

Acknowledgment is here given for the coöperation of R. F. Goudey, Los Angeles; C. W. Sopp, Pasadena; B. Burnson (East Bay Municipal Utility District); J. L. Perhab, Beverly Hills, in obtaining much of the data presented. The help of Prof. W. F. Langelier and the members of the Purification Division, San Francisco Water Department, is also acknowledged with appreciation.

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OPERATING KINKS*

By A. R. Todd

(Chemist, Wheeling Water Works, Wheeling, W. Va.)

At Wheeling we have a 20 million gallon rapid sand filtration plant. The source of supply is the Ohio River. This water varies from an acid condition of 9 parts per million, to an alkaline condition of 15 p.p.m.; from low river water of a turbidity of 5, to floods with a turbidity of 1450. Some one has stated that during extremely dry weather that 1 gallon out of every 4 in the Ohio River passing the plant has been through some city sewer. This sewerage plus the mine water and trade wastes makes a raw water at times most difficult to treat, and at other times very easy to treat. In other words, if conditions are just right, there is a natural coagulation from the iron sulphate in the mine water, and the natural alkalinity of the river. At such times it is only necessary to add a small amount of lime for corrective treatment. Chemical examinations of the Ohio River water have shown at sometime or other most of the elements found in the chemistry texts—zinc, tin and iron sulphates and chlorides from the pickling operations at the tin mill plants, copper and iron, nitrites and nitrates from the alloy plants. In addition manganese, magnesium, and many other elements together with phenols, creosols and naphthalene have been found.

PURIFICATION PROCESS

Since the river at most times contains sufficient iron sulphate, we find it only necessary to use lime to get perfect coagulation. When the river water contains none or very little iron sulphate, we add iron sulphate in sufficient quantities to amount to 0.1 grain per gallon per 100 parts per million of turbidity. In using the various materials in water treatment we have found that 2 pounds of carbon per million gallons, applied to water just as it is going on the filters, is as effective as 14 pounds per million gallons applied to the raw water, in so far as tastes and odors are concerned. Also that 2.8

* Presented before the Central States Section, August, 1937.

pounds of chlorine per million gallons of water applied to the finished water, is as effective as 7.8 pounds as prechlorination in sterilizing the water.

From November 1933 to November 1936, our purification process consisted of first adding ferrous sulphate if there was an insufficient amount in the river water, then lime, and operating the settling basins in tandem. We added ammonia water just as the water left the 1st basin, (We use aqua ammonia instead of anhydrous because aqua ammonia is about ⅓ ammonia and costs 3¢ per pound delivered, while anhydrous costs us 17½¢ per pound delivered). Chlorine was added just as the water was entering the 2nd basin; then the water was postchlorinated as a precautionary measure, using ¼ to ½ pound chlorine per million gallons. This method gave us fairly good results. The filter runs averaged 70 hours, our wash water use less than 1½ percent. The sand remained in good condition, but we did have six periods of from one to three days of taste and odor trouble. About the first of November 1936 we had a raw river condition so bad that people living ½ block from the aerators complained of the odor. From April 1936 until that time—some seven months—the water had been acid, then the water changed to the alkaline side. Evidently the change from acid to alkaline affected the animal life of the river, but would not account for the phenols, creosols and naphthalene. We took some naphthalene, put it in distilled water, shook well, then allowed it to stand 24 hours. This gave us a saturated solution of naphthalene in water. By taking 100 cc. of this solution, diluting it with 100 cc. of water, we got a 50 percent saturation. This compared exactly to the river water. Right here I want to digress a moment. This naphthalene and distilled water gave us an idea—in fact, two of them. We made dilutions of 1 to 2, 1 to 4, 1 to 8, 1 to 16, 1 to 32, 1 to 64, 1 to 128, 1 to 256, and tried them out on our various employees; we found that out of 12 people, nine detected the 1 to 64, two people the 1 to 32 and 1 to 128. So now we have a basis of knowing just how good our nose is. In other words we have a universal standard. If you do not have any naphthalene in the labratory bring some moth balls from home.

The second idea was right in front of our nose and we couldn't see it. When the river water was acid we had no trouble with tastes or odors—when it turned alkaline we did. We took 60 grams of phenol, dissolved it in 2000 ml. distilled water, divided into 2 parts. To one part we added ferrous sulphate; to the other part

some lime. Then we passed chlorine gas in these solutions. Here is the answer: In the one containing lime plus phenol—a resinous product formed, then a heavier liquid settled out and an odor developed that was anything but pleasant. A little of this resin floated in water and gave all the characteristics of water with the medicated odor or taste. Now the other sample containing the ferrous sulphate gave an entirely different effect. There were no resinous nor bad odors, but instead a rather pleasant, faint perfume-like odor. We then made up various solutions using raw water with phenol concentrations of from 1 to 10,000 to 1 to 1,000,000, with pH of 3.8, 4.4, 5.0, 5.5, 6, 6.5, and 7.2 respectively. We then introduced a solution containing ferrous sulphate with chlorine, shook it well, and allowed it to stand ten minutes for oxidation. Then lime water was added to bring the pH up to 9.3. The solutions were then filtered and tested for odor and taste. In all of the solutions from a pH of 3.8 to 5.5 no odor or taste could be detected. But at a pH of 6 to 7.2, characteristic odor and taste developed.

To make a long story short, raw water containing phenols or creosols and naphthalene, can be effectively treated with ferrous sulphate and chlorine at a pH of 5.5 or under, and will not develope the characteristic taste when lime is added later.

The best method we have found to accomplish this is to feed ferrous sulphate and chlorine together. For chlorinated copperas one feeds 1 pound chlorine to 7.8 pounds ferrous sulphate, but this is not what is required to oxidize phenols. The ferrous sulphate is needed to act as a carrier or possibly a catalytic agent. It is used as such in oxidizing organic compound with chlorine.

The ferrous sulphate answers three purposes: 1st, as a coagulant; 2nd, it lowers the pH of the raw water; and, 3rd, it acts as the carrier for the chlorine. So instead of feeding 1 pound of chlorine to every 7.8 pounds of ferrous sulphate to make chlorinated copperas, only feed 1½ pounds of chlorine per million gallons of water, regardless of the amount of ferrous sulphate required for the other two purposes, that is, as a coagulant or to bring the pH down. Since last November this has been our purification process. 15 minutes after the ferrous sulphate and chlorine is added, lime is added to bring the pH up to 9.3. Then in order to keep the filters in good condition, another pound per million gallons of chlorine is fed just as the water goes on the filters. This method cured our taste troubles last November and has been most successful to date.

LOW WATER TEMPERATURE RESULTS

Last winter we had 44 days when the raw water was at its maximum density and no difficulties whatever were experienced in getting the floc to settle out. We attribute this to the fact that we were feeding ferrous sulphate and chlorine together. During this time, the raw water varied in turbidity from 20 to 475 p.p.m., and the water coming over on top of the filters showed a turbidity of 3 p.p.m. or less. During previous winters, when the water reached the temperature of 39°F., we invariably had trouble with floc not settling properly in the basins and coming on the filters. Evidently, there is a difference in specific gravity of the floc.

CHLORINE TESTS

So much for the water treatment. Occasionally, we get raw water in Wheeling high in nitrites along with the usual ferrous and ferric salts and manganese. In making the ortho-tolidine test for chlorine we can at times get a test on the raw water equivalent to 0.5 parts per million of chlorine, and this condition will persist all through the plant. In fact, at such times one can set aside such water for weeks, and still get a test. When nitrites of from 2 to 4 parts per million are present, the ordinary tests for chlorine with ortho-tolidine or potassium iodide and starch are absolutely worthless, whether in the light or dark, or any other condition mentioned in the books. So the separation of chlorine, nitrites, iron and manganese, and cobalt, offers some difficulty. The ortho-tolidine test solution is made up using some hydrochloric acid. I had an idea that perhaps the manganese, when precipitated as the hydroxide, was converted to the oxide and thereby liberated chlorine from the hydrochloric acid. This was an erroneous idea, as ortho-tolidine solutions were made up using instead of hydrochloric acid, acetic acid or phosphoric acid, and the results were the same. We also found that you cannot filter water before making the ortho-tolidine test for chlorine. Filter paper or cotton takes out the chlorine. In the presence of a high nitrite, iron, and manganese, there is only one sure method of knowing whether the color developed is chlorine, and that is to set up a small all glass still and distil the sample. All of the chlorine comes over in the first 60 ml. The method is to take 100 ml. of the water, place it in the still and collect the distillate in a tube containing 1 cc. of the ortho-tolidine solution, and 39 ml. distilled water. The chlorine recovery is 60 percent with 0.1 part

per million chlorine. If you get a test showing 0.6 p.p.m. chlorine you know you had a 0.1 to begin with, or if you want to you can take the pH of the distillate on each successive 10 ml. portion, and from this you can work out your own table.

FILTER SAND

Now regarding filters. If the sand is in bad condition, do not prechlorinate. If you do you will have plenty of grief unless you start out with not over ¼ pound of chlorine per million gallons, and gradually build it up, expecting to take at least four months before getting chlorine through the filters. This statement also applies to bringing up the pH for corrosion control. You will have plenty of trouble due to sloughing off of the iron oxide in your mains, causing red water if you try to bring the pH up too rapidly. If the filters are slimy with organic matter, use ordinary common salt, not less than 1½ pounds to the square foot. Place this on the sand covered by a few inches of water, then lower the water and let stand 24 hours, then give a good wash. You will be surprised at the results. In fact, you will find it more effective than sodium hydroxide. The sardine canners use salt water instead of fresh water in washing the fish, because it is so much more effective in removing the slime. The high velocity wash still leaves us with filter troubles. The best method we have found to remove the hard spots and mud balls, is to take either a ¾ inch or a 1 inch hose attached to a length of 7 feet of 1 inch pipe. Instead of a nozzle at the end place a cap in which a ½ inch hole has been drilled. Use water at 90 to 100 pounds pressure. This gives plenty of water and will not disturb the gravel. One man can get over a 712 square feet filter in about two hours. We find that doing this twice a year keeps the sand in good condition, as evidenced by increased filter runs. We carry our finished water at pH of 9.3 even though the marble test indicates 8.8 to be the correct point. And the reasons are this: first, no complaints on red water except a few dead ends ending up a hillside; second, at 9.3 the water remains at 9.3 all over town; and third, at 9.3 less iron is dissolved from iron filings than at any other pH.

ELECTROLYSIS

Now regarding electrolysis, I venture to say that this is the cause of 99 percent of our red water trouble. This includes stray ground currents and grounding on our pipe lines. Summer before last we

had a number of complaints from an area about three blocks square in one of the better sections of the city. The hydrants were blown repeatedly and still complaints. So we went out and tested the pipe for stray currents. We found a flow of two milivolts going into the pipe. Then a street car passed and it went up to 50 millivolts. In order to find out just what was happening to the water we flushed the pipes and then took samples every two hours for 24 hours and noted what happene d.

	WATER AT START	AFTER 24 HOURS
		p.p.m.
Iron	None	5
pH	9.3	7.8
Phenolphthalein alkaline	6	0
Carbon dioxide	0	6
Total alkalinity	22	15

To prove that the cause was the stray currents, we set up an apparatus in the laboratory using direct current, ran tests using from ½ millivolt to 45 volts with same results—the higher the current the more rapid the results. We then went down 150 feet to the end of the filter gallery. We attached one of the wires here and the other in the laboratory. Water in this pipe behaved just the same. We assumed that while alternating current would have no effect, we tried it out anyway. Very much to our chagrin and surprise we got the same results. We do not have to worry about the effect on the mains, because of the small quantity of iron going into the solution. But those of us responsible for the quality of water should protest any kind of grounding on our water mains.

BACTERIA AND pH

Bacterial conditions of the raw Ohio River water at the Warwood plant are the plate count varies from 0 to 24,000 and coliform organisms from 0 to 1000 per ml. The plate count varies almost directly with the pH, at a pH of from 3.8 to 5.0. The count varies from 0 to 400 with an average of 80. At a pH from 5.0 to 5.5 the count varies from 100 to 800 with an average of 140. At a pH of from 5.5 to 7.2, the count will jump from 1000 to 24000 with an average of 2000.

LABORATORY WORK

In addition to our regular work the laboratory takes care of the needs of the city and county Board of Health. Most of this work is routine—such as milk, well water samples, etc. About a year ago the health commissioner decided that he wanted to know just how sanitary the spoons, forks, water glasses, beer glasses and ice cream dippers were. So since then we have made several hundred tests. The first lot of samples received showed plate counts as high as 200,000 on beer glasses, and 1,000,000 on ice cream dippers. Spoons and knives had from 0 to 2,000; forks from 10 to 10,000. Incidentally, the common eating fork lacks much to be desired as a sanitary eating utensil. Also, what we would term the better class restaurants and beer places showed the best results, and the so-called night clubs the worst. The method that we use might be of interest to you. We prepare in the laboratory the swabs for taking the samples. We use a medium size test tube, introduce the swab, close the end with a cotton plug and sterilize. Before the tube with the swab leaves the laboratory, we add ½ ml. of sterilized salt solution, made by dissolving 8.3 grams sodium chloride in one liter of water. The inspectors take these swabs, and on a beer glass they pass the swab once around the rim of the glass or both the inside and outside.

When the tubes are returned to the laboratory, an additional 3½ ml. of the sterilized salt solution is added to each tube. The tube is then well shaken, and 1 ml. plated in the usual way, and incubated at 37° for 48 hours.

MOLD SPORES

The past winter, as you all recall, was a mild one but I do not know whether or not this would account for a new trouble we experienced at the Wheeling plant in the contamination of the raw river water with mold spores. Counts of 100 per ml. were not uncommon. We found it necessary to make at least five plates per dilution on the raw water, in order to get a plate free from mold, so as to get an accurate plate count. If the mold count ran five per ml. or less, we had no difficulty with the coagulated plate counts; if over this, we did. When the plate counts were running over 100 molds per ml., we had on three separate occasions a mold count in the finished tap water, notwithstanding the fact that we were chlorinating at three separate points; that is, prechlorination, intermediate chlorina-

tion, and post chlorination. There appeared to be three separate kinds, one that simply covered the plate in all directions, another the formed an inverted cone which is a thing of rare beauty, and the third simply formed large colonies.

CARBON

Large quantities of gasoline are towed up and down the Ohio River. Last winter one of the barges got loose and rammed a bridge pier, and the gasoline covered the water at our intake pier. The water coming into the plant had a very strong odor of gasoline. Four pounds of activated carbon per million gals. completely removed this odor. In a small city near Wheeling, they attempted to use an old oil pipe line for conveying water, and had all kinds of complaints that the water had a kerosene taste. I advised four pounds of carbon per million gals. and this was a complete success. I am merely citing these experiences so if you ever experience these troubles, you may know what to do.

CHEMICALS IN WATER

A water works chemist during the day wages a constant battle to remove all the iron and manganese from the water, and hesitates in the use of copper sulphate for algae. In the evening he reads that in one section of Florida, large quantities of manganese are needed as a fertilizer. In an advertisement we read that eleven chemicals—iron, sulphur, phosphorus, boron, nitrogen, manganese, magnesium, copper, calcium, potassium, and silicon are needed in our gardens to produce perfect plants. Also, young chicks will not develop, but on the contrary will grow anemic and die, without small amounts of manganese, and that traces of copper in some form will cure other cases of anemia in the human being. The physician prescribes organic iron salts, and certainly there are organic salts of iron in the Ohio River. There is also nickel, and occasionally traces of cobalt are present, and many other organic and inorganic chemicals.

So, sometimes, we wonder, aside from being clear and sterile, just how pure a drinking water should be.

SUMMARY

Common ordinary salt was found more effective than sodium hydroxide in removing slime and other organic matter from filter beds.

Phenols and other taste and odor producing compounds can be successfully eliminated by the use of ferrous sulphate and chlorine at pH of 5.5 or under.

Electrolysis, due to stray currents and also grounding on water mains, is a major cause of red water troubles.

When raw water troubles arise in making chlorine tests, as well as in the presence of heavy nitrites, filter paper or cotton in filtering removes chlorine. The only satisfactory method found was to distill off the chlorine.

HOOD FOR FINISHED WATER OBSERVATION STATION*

By Carl Leipold

(Sup't. Water Filtration Plant, Winnetka, Ill.)

The observation station for the finished water reservoir at the Winnetka water filtration plant was installed for the purpose of satisfying the requests made by water plant visitors to view the finished water, and to assist the laboratory in turbidity control. Results obtained from the observation station showed that improvement in arrangement and method of observation was necessary.

In the original arrangement of the station, it was found that the person making the observation was obliged to bring the head close to the glass of the portlight, the nose touching the glass, and shielding the eyes with the two hands for protection against interference of outside light in order to obtain a satisfactory view of the water, especially on clear days. The breath of the observer was also exhaled on the glass. This method of observation, with the hands and the nose being in contact with the glass, and the exhaled breath on the glass, gave sufficient evidence of unsatisfactory conditions from a sanitary point of view, as well as improper light control for an impressive view and turbidity control of the water.

Experiments made with various equipment, showed that a hood placed over the glass portlight would serve the purpose with satisfactory results. Consideration was given to various telescopes but these proved to be unsatisfactory from a sanitary point of view and also involved attention for keeping the equipment clean.

The hood (Figure No. 1) was constructed of $\frac{1}{8}$ inch Masonite, having top enclosure and an opening on the side of size $16\frac{3}{4}$ inches wide by $17\frac{1}{4}$ inches high, the interior being painted black. This arrangement permits the face of the observer to be 6 inches above the glass portlight and assures an impressive view of the water with satisfactory turbidity control. The method of observation is to have the station light at OFF position, then to first place the head

*Contributed.

within the enclosure enabling the eyes to be accustomed to the dark enclosure, and finally to snap the electric light switch to ON position.

The improved arrangement and method of observation, has shown that very small amounts of suspended matter (readings below 1.0)

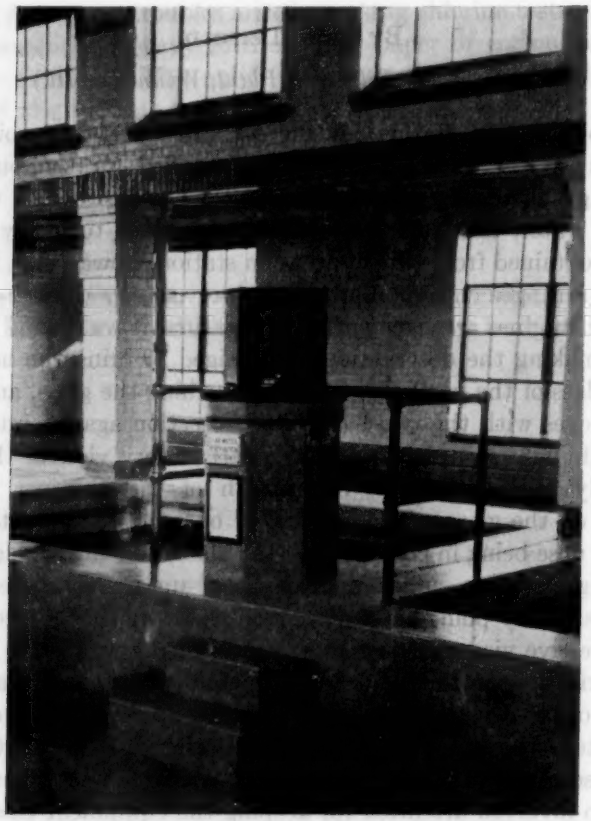

FIG. 1. OBSERVATION STATION HOOD DETAIL

can be detected; very little attention is needed for cleaning equipment; and, an added assurance of a correct turbidity reading at each observation regardless of surrounding weather or light conditions at any time of the day. For plant control, this station serves as an essential indicator of filtration results and filter conditions.

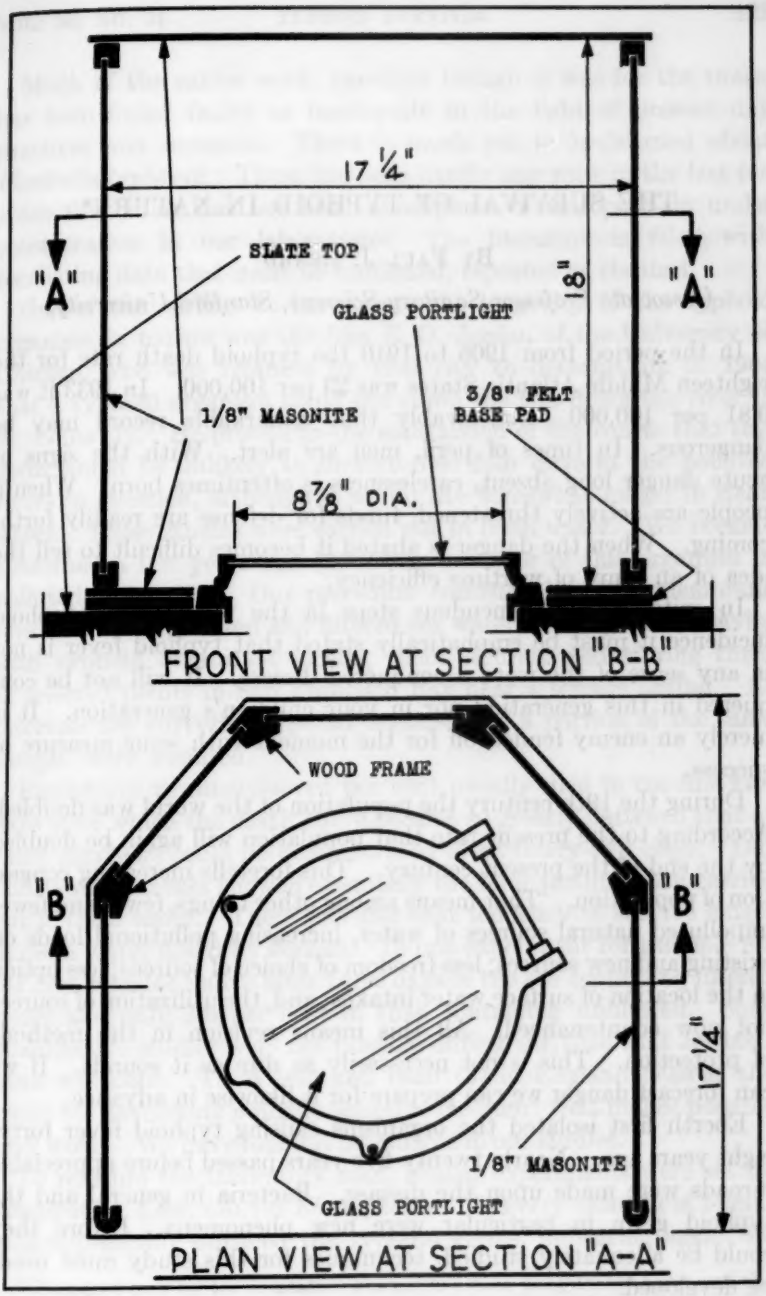

FIG. 2. VIEW SHOWING HOOD ON OBSERVATION STATION

THE SURVIVAL OF TYPHOID IN NATURE*

By Paul J. Beard

(Associate Professor Sanitary Sciences, Stanford University)

In the period from 1906 to 1910 the typhoid death rate for the eighteen Middle Atlantic States was 23 per 100,000. In 1933 it was 0.81 per 100,000. Conceivably that remarkable record may be dangerous. In times of peril, men are alert. With the signs of acute danger long absent, carelessness is oftentimes born. When a people are actively threatened, funds for defense are readily forthcoming. When the danger is abated it becomes difficult to sell the idea of an army of wartime efficiency.

In spite of the tremendous steps in the reduction of typhoid incidence it must be emphatically stated that typhoid fever is not in any sense of the word a conquered disease. It will not be conquered in this generation nor in your children's generation. It is merely an enemy fended off for the moment with some measure of success.

During the 19th century the population of the world was doubled. According to the present rate that population will again be doubled by the end of the present century. This foretells increasing congestion of population. That means among other things, fewer and fewer unpolluted natural sources of water, increasing pollutional loads on existing and new sources; less freedom of choice of sources; less option in the location of surface water intakes; and, the utilization of sources not now countenanced. All this means revision in the methods of protection. This is not necessarily as dire as it sounds. If we can forecast danger we can prepare for it likewise in advance.

Eberth first isolated the organisms causing typhoid fever forty-eight years ago. Nearly twenty-five years passed before appreciable inroads were made upon the disease. Bacteria in general and the typhoid germ in particular were new phenomena. Before they could be adequately studied, techniques for this study must needs be developed.

* Presented before the California Section, October, 1937.

Much of the earlier work, excellent though it was for the times, has been found faulty or inadequate in the light of present day practices and demands. There is much yet to be learned about *Eberthella typhosus*. There has been hardly any time in the last ten years that there has not been some phase of this problem under consideration in our laboratories. The literature is filled with conflicting data that must be confirmed, repeated or clarified.

One of the earliest to investigate the longevity of the typhoid organism in nature was the late E. O. Jordan of the University of Chicago, who was retained by that city to demonstrate in 1904 that a typhoid organism could not stand the trip down the river to St. Louis (1). He proved to the satisfaction of the courts that this germ would be unlikely to survive over four days in the polluted rivers in question, although it would live somewhat longer in clean water. These studies were carried out in fresh water. We became concerned a few years ago (2) in the behavior of this organism in polluted sea waters. Our procedure consisted in filling membrane sacs with both polluted and filtered sea water, seeding them heavily with typhoid organisms, closing them securely and storing them at selected points in San Francisco Bay near a sewage outfall. At intervals the surviving typhoid organisms in a portion of the same sample were counted.

Eighty-five to ninety-seven per cent usually died in the first two days after which the death curve became so greatly flattened that at times survivors could be demonstrated after thirty-two days. Duplicate experiments showed essentially the same picture when carried out in fresh water. There was also that tendency to survive longer in clean water noted by Jordan. In one experiment for example 8.2 per cent remained alive after three days in the sac containing filtered water and only 3.6 per cent in that containing unfiltered. Such quantitative studies were not possible during the time that Jordan's work was done. To Wilson and Blair (3) in England belongs the credit for developing a highly selective medium* that makes possible such work. We have found it invaluable in our studies.

To reassure the laboratory man, similar experiments were set up substituting *Esch. coli* for *Eb. typhosus*. In every case these organisms appreciably outlived the typhoid germs, indicating again the

* Prepared now by the Digestive Ferments Co. in dehydrated form and the author wishes to acknowledge their courtesy in supplying the media used in various experiments quoted here.

reliability of the colon group as a danger signal as far as that phase of relative longevity is concerned.

The question of typhoid survival in soils is of great interest to the waterworks man from the standpoint of watershed safety. With the insistence by the public on the recreational use of watersheds this becomes a greatly enhanced problem and one of increasing concern.

The reports along this line vary greatly. That is natural since experiments carried out in nature can never be exactly duplicated. Melick (4) observed survivals in soils infected with excreta, over periods varying from 25 to 41 days. He also noted that young strains were less resistant than old. In feces spread on soil, Galvagne and Calerin in 1908 (5), found living organisms on the surface after ten days and after 40 days in the deeper layers. Mair (6) claims to have isolated typhoid bacilli from 84 days. Such survival times as these would constitute long periods of potential danger on watersheds, particularly those exposed to intermittent rains.

Some of our efforts (7) have been directed toward learning certain of the factors that influence the ability of these organisms to maintain themselves in soil. In these experiments the various soils used were infected and placed out of doors in screened containers where they were left exposed to rain, sun and wind until the conclusion of the experiment. At suitable intervals samples were removed to the laboratory and the survivors counted. Two series of experiments, one during the rainy season and one during the dry season, have been carried out with a variety of soils,—adobe, adobe-peat mixture, loam, and peat.

The longest period of survival occurred with loam during the rainy season, demonstrating a survival of over 50 days. In the same soil during a dry season the survival was between 21 and 28 days. Inspection of our data showed that survival varied with the type of soil, the most significant single factor apparently being the ability of that soil to retain moisture. In sand during the dry season the survival varied between 2 and 7 days depending on humidity and temperature. There were definite indications also that pH may have a rôle. Thus, in peat, infusion which has a pH near that lethal for typhoid, the maximum period of survival regardless of season was 24 hours. Other conditions being equal, the lower the temperature, the longer the survival. Death rates at the surface were greater on clear sunny days than on cloudy days even though temperature and humidity were comparable.

Summarizing our data to date we find the moisture retaining power of a soil, the temperature, humidity, rainfall, and pH to be especially significant factors in determining the length of time that typhoid organisms will survive in it. Another possible factor that we have not been able to evaluate is effect of other soil organisms.

Definitely related both to soil infection and stream pollution is the fate of the typhoid bacillus in the various systems of sewage disposal and treatment. Relatively few studies have been carried out in this field. The technical difficulties have been very great and until recently satisfactory quantitative studies have well nigh been impossible. The earlier work of Jordan and his associates previously referred to, dealt to a certain extent with this phase. Bruns and Sierp (8) found living typhoid organisms in settled activated sludge after six hours aeration although the supernatant fluid indicated a 96 percent decrease. Wilson and Blair and later Ruchhoft (9) were able to isolate living typhoid organisms from sewage. The latter, in experiments with activated sludge found an 86 per cent reduction in the sludge-fluid mixture after five and a half hours aeration. Settling brought about a 5.8 per cent removal of these or a total reduction of 99.2 per cent. Heukelekian and Schulhoff (10) studied the death rates in the activated sludge process with essentially the same results, indicating, however, a temporary increase in numbers.

During this last year Green of Oregon, working in our laboratories undertook further investigations along this line (11). He was able to isolate and prove the identity of typhoid organisms on several occasions from raw Palo Alto sewage, a city with no carriers recorded and only two or three cases during the last ten years.

Under conditions simulating those of septic tanks he found that 98 per cent of the typhoid organism introduced therein died off in four days, but even after 27 days there were still at least 10 living typhi per cubic centimeter. Again the lower the temperature the longer the survival. PH also was again a significant factor.

Laboratory size activated sludge plants holding 4 liters were constructed, brought to maturity, and seeded with typhoid cultures. There occurred a 96 per cent reduction of the organisms with six hours aeration and 99 per cent after eight hours. With a view toward determining something of the mechanism of this reduction, the sludge and raw sewage were heated sufficiently to eliminate all but the most resistant sewage bacteria and protozoa before being inoculated with the typhoid culture. The control was as before and as

before the same reduction occurred. In the heated batch, however, instead of reduction there was growth and the increase in numbers continued for three days, when there followed an abrupt decline. This indicates at least that mere aeration in itself exerts no killing effect.

From the technical standpoint the most difficult problems were the studies on death rates in a trickling filter. There are certain problems involved in spraying hundreds of gallons of *Eb. typhosus* laden sewage into the air. The neighbors are apt to complain if an epidemic is started. Further, we ourselves were anxious to complete studies. Typhoid fever lays one up quite a while and these studies were extended over several weeks. We were frankly relieved when the experiment was completed and the incubation period for the disease had passed.

Dosing at rates up to 6.6 million gallons per acre per day (constant dosing) indicated a 99 per cent reduction in numbers. The B.O.D. reduction was 80 per cent. At 12 million gallons per acre per day rate the typhoid reduction was 95 per cent.

With an intermittent dosing cycle, 1 million gallons per acre per day rate, two minutes dosing and six minutes resting, with 5.5 million organisms per c.c. being poured onto the filter, we were never able to demonstrate more than 30 typhi per c.c. in the effluent, a remarkable percentage reduction. Composite samples collected at 0.5, 1.0 and 2.0 hour intervals contained only 10 organisms per c.c. in each instance. At a 2.8 million gallons per acre per day rate, five minutes dosing and 10 minutes resting, a 99 per cent reduction was indicated.

Another phase of typhoid survival concerning which there may exist some misunderstanding concerns that of survival in freezing or frozen waters. We hear the statements that water tends to purify itself upon freezing and that the ice produced even from highly polluted waters is likely to be innocuous. These are generalizations that approach the truth, but which must be accepted with proper understanding of the mechanism involved.

First it should be kept in mind that the purification that takes place during freezing is not necessarily due to the death of the organisms. True, the large percentage of the pathogenic organisms do die during the first few hours, but, as we have shown, that phenomenon may be considered normal. It occurs whether the water is freezing or not. The bacterial removal from ice is more of a mechan-

ical removal. As ice forms, a sort of sludge is developed through decrease in solubility of various constituents of the water and the organisms are precipitated with the sludge.

Now if the freezing takes place very rapidly this sludge may be entrapped instead of being precipitated in which case the ice would remain infective. We have been able to demonstrate such behavior experimentally. Further, when freezing took place at such a rate that the usual purification occurred it could be shown that small twigs, leaves or other debris frozen in the ice retained large numbers of viable typhoid organisms. The precipitated sludge remained infective at times as long as five months and when the ice was allowed to melt, the water again became infected. This sequence of events might be logically expected to occur in a reservoir during a spring overturn.

CONCLUSIONS

In all of the experiments cited we observed that the vast majority of the pathogens are quickly eliminated. Such observations should not be allowed to lull one into a false sense of security. We are not concerned with the percentage that die, but with the small percentage that survive. Our objective is not to render water less dangerous, but to render it safe. It is no consolation to the typhoid fever victim to philosophize that the water he drank was 99.99 per cent pure.

We do not offer these experiments under the illusion that they constitute revolutionary discoveries. Rather they are selected to call attention to the problems that still exist and offer concrete evidence as to the efficiency that can reasonably be expected from certain of our protective processes. There remain many other problems to be solved in connection with the behavior of typhoid in nature. The recent work of Heathman and her associates (12) offers new and startlingly different problems in resistance. They report considerable variation in the resistances of freshly isolated strains of *El. typhosus* to chlorination, with the further indication that certain strains may be more resistant to such treatment than the coliform groups. This is an exceedingly important pronouncement which should be extensively investigated for confirmation. This again illustrates the possibility that some of our procedures may be based upon dangerous assumptions.

Again there is the unpleasantly suggestive work of Mudge and Smith (13) indicating the possibility of "reversible death" occurring

at times following chlorination. These latter observations have recently received substantiation in our laboratories although at the moment we are unwilling to commit ourselves with an interpretation.

Time does not permit further discussion or listing of the problems that at present and in the future confront water technology. But in view of these data, experiments, and suggestions cited, I should like to close with the reiteration that water-borne typhoid is yet far from a conquered disease and that from the standpoint of menace it is just as real as it was fifty years ago.

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SILVER ION AND AMMONIATED SILVER ION AS STERILIZING AGENTS IN A SWIMMING POOL*

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The phenomenon of oligodynamy, specific activity exerted by minute concentrations of certain metal ions was discovered by Nageli in 1893 (1). Aside from the use of traces of copper to inhibit algae in water supplies, little attention was given to practical applications until 1929, when Krause (2) showed that the oligodynamic action of silver could be utilized in water sterilization. Following this the Katadyn process (3) was developed for introducing the silver ion electrolytically. This process is now used to some extent abroad and in this country for sterilizing drinking water, swimming pools, and industrial water supplies.

In view of the apparent practical success of the electrolytic process the question arose as to whether or not similar concentrations of silver ions from salts introduced into the water might not be equally effective. This has been studied under laboratory conditions by various investigators (4). Essentially no difference in effectiveness was found when the silver ion was introduced by salts, from massive metal, or electrolytically. The concentration of silver ion required to effect sterilization varied from 0.1 to 0.7 p.p.m., according to the amount of contamination and impurities in the water. A recently developed method for determining minute amounts of silver (5) facilitated the following investigation of the effectiveness of silver salts in controlling bacterial contamination in a large swimming pool.† Since the method is time consuming, determinations were not made as often as the bacteriological tests. A colorimetric method now in course of development is expected to overcome this difficulty.

* Contributed Record of Research.

† The swimming pool, of 280,000 gallons or approximately one million liters capacity, in the Men's Gymnasium, Oregon State College, was made available through the courtesy of Dr. Claire V. Langton, Director of Division of Physical Education, and Mr. E. P. Jackson, Superintendent of Buildings.

Only chemical and bacteriological phases are discussed here. Public health considerations other than sanitation may lead to a restriction of the use of silver in water supplies and swimming pools, just as the food and drug laws may restrict silver additions to beverages (6) and food products.

Following the summer session of 1936 the pool was cleaned and filled with fresh water from the city mains. The city water comes from mountain streams and is chlorinated as it enters the mains. The chlorine sterilizing equipment of the pool, which is used in addition to city chlorination, was disconnected but the water was filtered in the usual way. Constant circulation was maintained, effecting complete turn-over of the pool volume every eight or nine hours. Samples for bacteriological analysis and silver determinations were taken from the four corners of the pool. Since the quadruplicates checked closely, only averages are reported here. The bacteriological examination included the standard plate count and lactose broth fermentation tests for the coliform group. Plates were poured and tubes inoculated immediately at the pool, thus avoiding any residual action in the sample bottles.

EXPERIMENTAL RECORD

As a preliminary test the pool was first inoculated with broth cultures of *Escherichia coli*, *Pseudomonas aeruginosa*, and *Bacillus subtilis*. After allowing three days for complete mixing and for proving the absence of any inhibitory substance in the water, samples were taken for bacteriological analysis. Immediately following this, 250 grams of silver nitrate, equivalent to 0.15 p.p.m. silver ion, assuming 100 percent ionization, were added in solution from three 10-liter bottles by siphons delivering immediately in front of the three circulating inlets. The rate of delivery was so adjusted that one hour was required for delivery of all the silver solution.

Before all the silver solution had been added the pool water began to turn dark, changing from the normal blue to green and finally almost black. Although the ortho-tolidine test was negative the chlorinated city water contained enough chloride ion to react and a semi-colloidal cloud of silver chloride could be observed where the silver nitrate solution discharged into the water. While the silver concentration of 0.15 p.p.m. is within the solubility of silver chloride, the relatively high local concentration of silver ion at the points of introduction reacted with the chloride ion and before distribution

in the pool could effect solution, photo-decomposition occurred. The resulting black colloidal suspension was removed by the filters in three days. This, of course, removed much potential silver ion and bactericidal agent. Absorption of silver ions probably by organic matter in the filters, further reduced the effective concentration with time.

The results presented in Table I show that the introduced silver reduced the bacterial plate count from 15,000 to 350 in one-half hour, and thereafter up to six hours exerted slight additional bactericidal effect. Subsequently the count increased, following the marked decrease in silver concentration. In all cases *Pseudomonas aeruginosa*, as evidenced by colony appearance, was the predominating organism on the plates.

TABLE I

Bactericidal effect of silver nitrate added to inoculated swimming pool

HOURS AFTER ADDITION OF ALL SILVER NITRATE	TOTAL SILVER AS SILVER IONS AND SILVER CHLORIDE	PLATE COUNT	COLON-AEROGENES GROUP
	p.p.m.		
0	0	15,000	Present
½	0.15	340	None
2½		225	None
3½		200	None
4		300	None
6	0.077	300	None
8	0.054		
9		450	None
51	0.032		

In an attempt to avoid coloration of the water and resulting loss of silver due to photochemical action on the silver chloride it was decided to make a silver nitrate addition at night, thus providing more time for solution of the colloidal chloride before light entered as a complicating factor. Nine days after the first addition, silver nitrate equivalent to 0.3 p.p.m. silver was added in the late evening. The pool was clear early on the following morning but shortly turned greenish and was black again by nine o'clock, indicating that the rate of solution of the silver chloride was too slow. As in the first experiment, the plate count was markedly reduced for the first few hours, falling from 540,000 to 2,000. It then increased slightly following the appearance of the coloration.

Since addition of the silver at night did not solve the pool darkening problem, a slower rate of adding was next tried in an attempt to keep the local concentration within the solubility of silver chloride. For this third addition of silver nitrate, again equivalent to 0.3 p.p.m. silver, the siphons were regulated at such a slow rate that 24 hours were required for the introduction. The pool remained clear for two days and then darkened. The reduction in number of viable bacteria was most effective, the count being reduced from 3400 to 10 in 24 hours. Due to the apparently inevitable darkening, however, the use of silver nitrate was discontinued and with the opening of the college year chlorination was employed.

By preliminary laboratory experiments it was found that in the presence of a slight excess of ammonia, ammoniated silver ion could be added to chlorinated water without incurring subsequent formation of a silver chloride cloud and photochemical darkening thereof. It was also found that ammonated silver ion is approximately equal to silver ion in bactericidal effect. In test solutions inoculated with a mixture of *Escherichia coli*, *Pseudomonas aeruginosa*, and *Bacillus subtilis*, giving a total concentration of 840,000 cells per c.c., all bacteria were killed within 15 minutes by 0.15 p.p.m. silver either as silver ion or ammonated silver ion; 0.025 p.p.m. silver in either form greatly reduced the number of viable cells but did not effect sterilization in one hour.

When the next opportunity for experimenting with the pool was presented during spring vacation chlorination was discontinued and 250 grams of silver nitrate dissolved in ten times the theoretical amount of ammonium hydroxide required to form the ammoniated silver ion was siphoned in at the three inlets from 10 liter bottles. No visible silver chloride cloud was formed and no subsequent darkening of the pool occurred. The plate and coliform group counts were maintained at zero on tested amounts by this addition for more than a week.

Experiments, as yet incomplete, were made to determine the minimum amount of ammonium hydroxide needed to prevent formation of silver chloride cloud. A ratio of one liter of 28 percent ammonia water to 100 grams of silver nitrate was used on several occasions without cloud formation and darkening, but a satisfactory minimum has not been established.

As long as the pool remained sterile it was decided to add no more chlorine. It was reopened for use during the spring quarter and was subjected to heavy bathing loads without further chlorination.

The ortho-tolidine test was negative and apparently the only sterilizing agent present was silver ion. Except for the first few hours following any addition of silver the actual concentration of silver in the water was found by analyses to average only 0.05 p.p.m.

Frequent bacteriological tests showed that the pool was kept practically sterile for 18 days. The count then suddenly became excessive. Further additions of ammoniated silver nitrate were only sporadically successful in controlling this excessive count, although at no time was the coliform group test positive. Finally by establishing a higher silver concentration in the pool, these more resistant organisms were killed. In all cases the excessive counts were due almost exclusively to a small gram negative rod producing yellow pigment. It is of interest to note that this organism is similar to the chlorine resistant organism isolated from this same pool and described by Charlton several years ago (7). Furthermore, a similar organism has caused excessive counts in the chlorinated pool of the women's gymnasium on this campus. Mallmann (7), studying electrolytic silver sterilization of a swimming pool also found yellow pigmented silver resistant bacteria and reported that silver appears slower than chlorine in germicidal action. While definite bactericidal properties of silver have been demonstrated, further studies are necessary to determine factors which at times limit its effectiveness in swimming pools.

THEORY OF SILVER STERILIZATION

Several theories have been suggested to explain the bactericidal action of silver ions in water. Theories of adsorption or combination between silver ions and bacteria have been proposed. Closely allied to the combination theory is the electrolytic cell theory in which electrochemical action is thought to take place between ions and bacteria. If such is the case the oligodynamic metal ions must oxidize the organic bacterial cells. Silver ions are easily reduced to metallic silver by organic matter. The silver ions are then the oxidizing agent or the electron donor and the bacterial cells may be considered as the reducing agent or electron acceptors. Equations for the reaction might be similar to action of a tartrate and silver ion.

$$Ag^+ + \text{Tartrate} \rightarrow Ag + \text{oxidized Tartrate}$$
$$Ag^+ + \text{Bacteria} \rightarrow Ag + \text{oxidized Bacteria}$$
$$108^+ \text{ (protons)} \qquad 108^+ \text{ (protons)}$$
$$107^- \text{ (electrons)} \qquad 108^- \text{ (electrons)}$$

The action of silver ion as an oxidizing agent can be likened to the indirect oxidation by chlorine.

$$Cl_2 + H_2O \leftrightarrows 2HCl + (O)$$

$$(O) + \text{Bacteria} \rightarrow \text{Bacteria—O}$$

16^+ (protons)	16^+ (protons)
16^- (electrons)	18^- (electrons)

In both cases the bacterial substance is the reducing agent or electron donor.

This theory is substantiated by the fact that only those elements below hydrogen in the electrochemical series (that is, those readily reduced metal ions) are bactericidal in low concentration; and, by the fact that certain organisms show similar tendency to resist moderately high concentration of either silver or chlorine.

SUMMARY

(1) Bactericidal effect was produced by 0.15 p.p.m. of silver added as silver nitrate to a 280,000 gallon swimming pool artificially inoculated.

(2) Silver chloride cloud formation and subsequent darkening resulting from addition of silver nitrate to water containing chloride ion was prevented by adding the silver nitrate in ammonium hydroxide solution.

(3) Ammoniated silver ion was found approximately equal to silver ion in bactericidal effect.

(4) Silver nitrate, dissolved in ammonium hydroxide, was only sporadically effective as a sterilizing agent when the pool was subjected to bathing loads.

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INDUSTRIAL WATERS IN CANADA*

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INTRODUCTORY

The systematic study of domestic and industrial waters actually used or available for use in Canada, more especially of those of industrial importance, was begun in 1934. An account of the work done during the years 1934 and 1935 has already been published as Interim Report No. 1 (1). The present report, termed Interim Report No. 2, deals with the results obtained in 1936.

Reference should be made to Interim Report No. 1, for full details regarding the general program, as well as a discussion of the character of waters, a brief review of previous investigations, and an outline of the methods of analysis chosen for this investigation.

The procedure followed has been:

1. Sampling and complete chemical analysis of surface waters that are of industrial importance.
2. Sampling and partial chemical analysis of civic waters.
3. Obtaining information from the owners of plants, in which large quantities of water are consumed, concerning difficulties encountered in manufacturing processes that have their origin in the quality of the water used.

The Population Factor in a Survey of the Hardness of Water. The United States Geological Survey's investigation of the industrial utility of public water supplies comprises, analyses of the water supply at 670 centers of 20,000 inhabitants and over, including, however, several smaller communities so as to obtain better distribution of results over the less densely populated states. The 670 places contain 46.2 percent of the total population of the United States (2).

As Canada has only 31 places of 20,000 inhabitants or more, the adoption of this, as the minimum unit in a survey of the distribution of hardness in Canadian waters, would furnish inadequate data. For Eastern Canada, therefore, a population of 3,000 or more has

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been chosen and for the West 2,000 or more, giving a total of 235 communities, representing 45.5 percent of the population of Canada. The distribution, expressed in percentages of the total of each province, would be: for Nova Scotia, 36.8 percent; New Brunswick, 27.8 percent; Prince Edward Island, 18.3 percent; Quebec, 56.9 percent; Ontario, 56.7 percent, making a total of 53.2 percent of the population of Eastern Canada as far west as Sault Ste. Marie. In the Maritime Provinces, the inclusion of towns of 2,000 inhabitants would not raise the ratio of the population served appreciably, but in Western Canada it is advisable, not because it would make much difference to the ratio, but because it would give a better geographical distribution of the varieties of industrial and civic waters for these large areas. The ratio of the population of the whole of the western provinces so included would be 38.2 percent; of the province of Manitoba, 36.7 percent; of Saskatchewan, 17.3 percent; of Alberta, 28.5 percent, and of British Columbia, 53.9 percent.

These calculations are based on the 1931 census, with the exception of some larger cities, and of service adjacent to smaller towns and communities, for which calculation is based on the total number of persons served, in accordance with data obtained from the Engineering and Contract Record.

PROGRESS MADE DURING 1936

The area over which operations were conducted in 1934 and 1935 was considerably extended during 1936. Check samples of civic waters were also collected and the regular sampling of surface waters at key stations in Quebec and Ontario was resumed. Water at these stations is sampled yearly, whenever possible at high, mean, and low gauge, so as to determine seasonal as well as yearly variation in the composition of the more important lakes and arterial waters of Canada.

In 1934 and 1935, the waters studied comprised those of Ontario south of the Ottawa river and Georgian bay, and those of all Quebec as far east as Riviere du Loup. Complete analyses were made of 48 samples of surface water, and of 189 samples of waters supplying cities and towns having populations of 3,000 or more; determinations were made of hardness, alkalinity, calcium, and magnesium.

During 1936, the investigation was extended to embrace the provinces of Nova Scotia, New Brunswick, Prince Edward Island, that part of Ontario lying between Georgian bay and Sault Ste.

Marie, and the rest of Quebec, thus completing Eastern Canada as far west as Sault Ste. Marie, except for the mining areas north of lake Nipissing. Samples were collected of 77 municipal waters and 26 surface waters.

Thus, in Canada during the three years the investigation has been in progress, 340 samples of water have been collected and analyzed; these in many cases being but single samples, whereas in the United States similar work has been done for over forty years, 800 water supply papers have been published and many thousands of analyses made. In compiling results relative to the distribution of hard waters in Canada, the method employed, follows as closely as possible that used by the United States Geological Survey, as it is obviously desirable that results in the two countries should be comparable.

Many of the waters have been analyzed for mineral content for the first time. Some analyses have been checked and yielded fairly concordant results, but others have shown appreciable divergences in concentration and composition. Many analyses, especially from the Maritime Provinces, have not been checked.

As few official records of analyses of important Canadian water supplies are available, recourse has had to be made to independent sampling and analysis. In only a few waterworks in Canada are complete analyses of the water made, laboratory work being usually confined to determining quality, color, turbidity, etc., so that proper control may be exercised and the product be a clear, sparkling water, safe for human consumption.

For an adequate report on the quality of water supplies, many samples have to be taken and analyzed. The quality of a body of water, however large, differs from that of other mineral deposits because, besides being subject to seasonal changes in composition, due to concentration, dilution, and absorption, slow changes may take place from year to year.

Ground waters appear less liable to seasonal variation in composition, but exceptions to this are not rare as in the case of some well waters of the city of London, the composition of which is very unstable and subject to change at frequent intervals.

As the investigation progresses and more samples and analyses are available, changes may, therefore, be expected in the composition of the waters reported upon. The development of new water supplies as cities and towns grow larger or include new towns in their populations may also influence the composition considerably.

For reporting the hardness of waters in Eastern Canada, the following scale of hardness has been adopted:

Total hardness in parts per million, expressed as calcium carbonate	
1 to 60	Soft water
61 to 120	Medium hard water
121 to 180	Hard water
181+	Very hard water

Subdivisions of these may be made in case the hardness approaches the limits set; for example, water of 63 hardness may be termed medium hard to soft; of 122, hard to medium hard, etc.

The Maritime Provinces. Of two samples collected in Prince Edward Island one shows medium hard and the other hard water; of twenty samples from Nova Scotia, only one was hard, the others soft to very soft; and of twelve samples from New Brunswick, five were medium hard, although two of these were very close to soft, and seven were soft waters.

Only three surface waters were sampled in the Maritime Provinces, two from the St. John river at Edmundston and at Woodstock, and one from the Nipisiguit river at Bathurst. The reason for so few samples is that most rivers in Nova Scotia and New Brunswick are of little or no industrial value because towns, cities, and industries are usually situated on the estuaries of rivers and, owing to the tide, brackish waters extend twenty miles more or less upstream. The industries obtain their water supply from lakes and springs, many enterprises having their own water service.

In visiting manufacturing plants, it was learned that in general the waters in Nova Scotia and New Brunswick are soft, so that operating problems due to the quality of the water are usually caused by corrosion, in some cases by color only, and in a few by algae. Algae are objectionable as they impart an unpleasant (usually fishy) taste to the water, making it unfit for drinking and for the manufacture of food products. This trouble usually occurs in smaller lakes, more or less stagnant, and can easily be corrected by addition of copper sulphate. The chemical is placed in cheesecloth bags and trailed behind a motor launch, traversing the lake in several directions, to obtain a uniform distribution. Two such treatments during the latter part of the summer months usually suffice.

Corrosion, however, is more serious and for some time past has been the subject of extensive investigation by the large manufactur-

ing concerns. The corrosive action is due mainly to acidity caused by dissolved gases, oxygen and carbonic acid, which together form a very active agent. Experiments are still in progress for correcting such waters either by raising the pH value by addition of lime, soda, or tri-phosphate of soda, or by removal of the dissolved gases by preheating the water. The latter is considered to be applicable especially to boiler waters. No method of correction of industrial waters has so far been adopted which may be considered fully effective.

Quebec. Check samples for Quebec taken in the early part of the summer, with few exceptions, show only smaller changes in composition.

Quebec waters are preponderantly soft except those supplies drawn from the St. Lawrence river, which is a medium hard water, and even among the ground waters the hard types seem to be more the exception.

Ontario. Samples in Ontario collected during the latter part of the summer tend toward somewhat higher hardness figures as compared with samples of 1935 and 1936. This is probably due to a series of drought years, which has lowered the levels of most supplies.

The northern and eastern parts of the province have soft to medium-hard waters. South of Georgian bay, lake Simcoe, and a strip following the St. Lawrence river, the water is very hard to hard.

THE BASIN OF THE GREAT LAKES

Surface Waters. Surface waters collected in Quebec province represent mean gauge; for Ontario, low gauge, in fact, abnormally low, which is especially noticeable in the concentration of the mineral constituents of the waters, increasing thereby the hardness, chlorides and alkalies. This also is generally true but to a much less degree for the whole of the Great Lakes system.

Figure 1 shows the variations in hardness of the waters of the Great Lakes basin, computed from averages of two to eight samples, a number insufficient to afford as reliable an average as those published by the United States Geological Survey, that seldom represent less than twelve and usually twelve composites of daily samples. It is, therefore, likely that, as more samples and analyses are available, these figures may be subject to adjustment.

Variations in the mineral composition of these large bodies of water, judging from the comparative analyses, are not extreme and the diagram may, therefore, be regarded as showing the trend of the

variation of the total hardness for the waters of the Great Lakes Basin through its course to the sea.

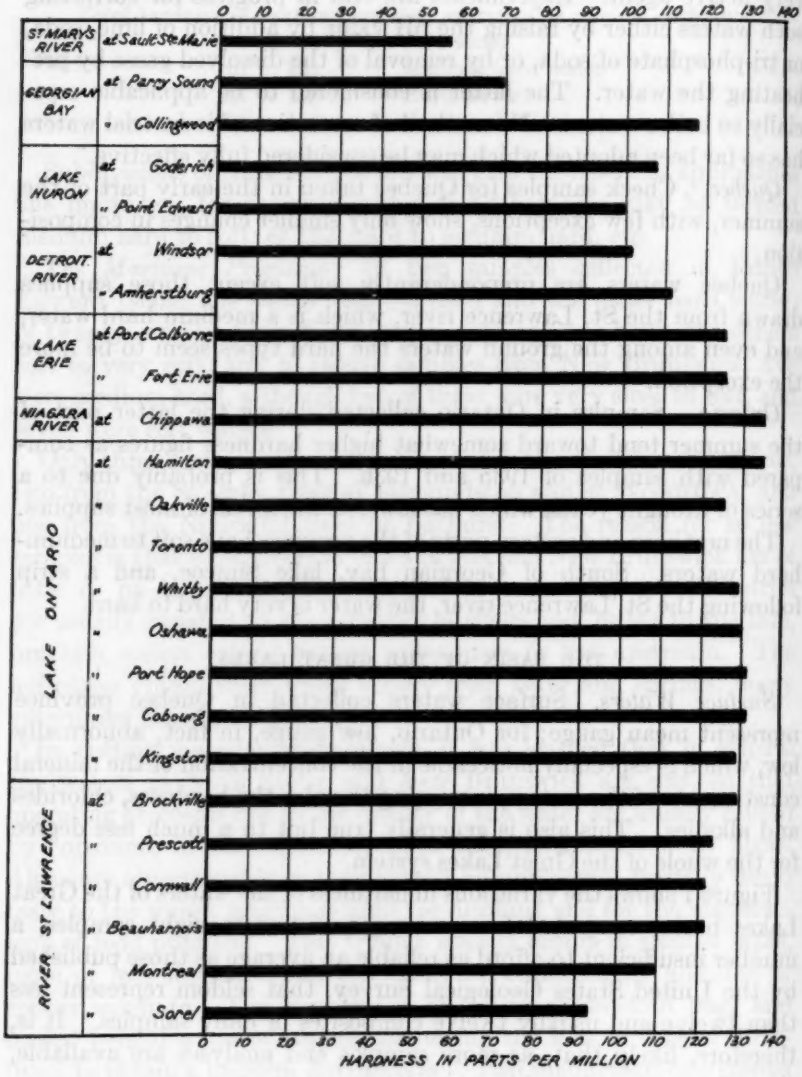


Fig. 1 Diagram showing variations of total hardness in parts per million of the waters of the Great Lakes basin.

The Lake Superior water is represented by analyses of samples taken from St. Mary's river above the rapids at Sault Ste. Marie, and has a hardness of 56.0 p.p.m.

The United States Geological Survey reports for twelve composite samples taken for each month during the year 1924 a hardness of 46.0 p.p.m. (3). Both figures represent the same classification as regards hardness.

The Lake Superior water is diverted in three directions, to Lake Michigan, with which this report is not concerned, to Lake Huron, and through the North Channel to Georgian bay. The water running through Lake Huron receives tributaries of water from the State of Michigan of very high hardness, noticeable already at St. Ignace, Michigan, where there is an increase in hardness to 82 p.p.m. (3). Mention may be made of the tributaries, Shiwassie river, Cass river and Flint river, which drain into Saginaw bay and have hardnesses of 221, 178, and 298 p.p.m. respectively (2).

The water flowing in the Georgian bay receives from its northern tributaries waters softer than that of Lake Superior, as in the case of analyses tabled for lake Nipissing where a dilution would be expected. This dilution is counterbalanced at the southern end of Georgian bay by the tributaries, the Severn, draining Simcoe and Couchiching lakes of hard water; and the Sydenham and a number of smaller tributaries of very hard water, increasing the hardness of the water considerably.

At Collingwood the Georgian bay water has a hardness of 117 p.p.m.; flowing north it mixes with the softer water and at Parry Sound it has a hardness of 70 p.p.m.

The mixture of the Lake Huron waters of various concentration appears to be fairly well accomplished at Goderich, where there is a hardness of 107 p.p.m., as from there on the decrease from 107 to 100 p.p.m. at Point Edward is not very appreciable.

Detroit river at Walkerville shows only a slight increase, in fact so slight that the difference might be due to analytical error. Nevertheless analyses of samples taken at Point Edward and at Walkerville at twelve hours' interval show consistently slightly lower total hardness for Point Edward water, which should be correct, as tributaries to the St. Clair river, notably the Thames river, although of comparatively small volume, have very hard water.

At Amherstburg there appears an increase of 10 p.p.m., which may be expected from the very large quantities of effluence and the heavy discharge of factory waste from the densely populated districts.

Lake Erie shows a further increase in hardness, samples taken at Port Colborne and Fort Erie having a hardness of 125 p.p.m. Tributaries to Lake Erie have all very hard waters, the Huron river, Woolf

creek, Raisin river, and Maumee river having 298, 295, 245, and 207 p.p.m. total hardness (3) respectively and even harder water being received from drainage and tributaries from Lake Ontario, for example, the Grand river, which varies from 275 to 400 p.p.m.

The peak in concentration for hardness appears to be reached at Niagara Falls and Hamilton, with 136 p.p.m. It is true that this is supported by analyses of two samples only from each place, but eight samples taken along the Welland Canal, where the water flows more slowly through the same formation as the Niagara river, gave variations from 128 at Port Colborne to 143 at Merritton, with an average of 136 p.p.m. Analyses furnished by the water works of the city of Hamilton from which the total hardness has been calculated yield even higher values.

Lake Ontario shows a slight dilution at Oakville; this hardness appears uniform for the whole of the Canadian shore line of the lake as far as Prescott. The dip in concentration of hardness at Toronto would not appear justified in view of the very hard waters from the Don river, from industrial waste, etc., from such a large city, which should cause the opposite. The figure, however, is an average of five analyses giving hardness ranging from 98.5 to 128.5 p.p.m. and, had it not been for one analysis much lower than any of the others, it would have been appreciably higher. Another reason may be that samples collected at the Center Island water works at Toronto were taken at a depth of 90 feet from heavy low-level currents that may not be affected by waters from adjacent tributaries, industrial waste, etc. Depth samples from other key stations are usually taken at about 25 feet or less, depending on the depth of the lake or river.

The River St. Lawrence. At Prescott there is a drop of about 7 p.p.m. in hardness which is maintained until the Ottawa river joins the St. Lawrence. No tributaries of importance come from the Canadian side, but from the State of New York there is an appreciable drainage and the waters are medium to very soft. The Oswegatche river at Ogdensburg has a hardness of only 50 p.p.m. (3).

A true average of the Montreal water is difficult to obtain unless numerous samples are collected throughout the year. The hardness of the nine samples so far collected and analyzed ranged from 74 to 124 p.p.m. This great range in hardness is due to the soft and colored water of the Ottawa river, the largest tributary of the St.

Lawrence, joining it at lake St. Louis. The two waters do not mix as readily as might be supposed and may run for many miles before complete mixture has been accomplished.

A parallel case, often quoted, is that of the blue Mississippi and the white Missouri, which, after joining, run for miles with a sharp dividing line between the two waters and much farther before a complete mixture has been attained.

The bulk of the water used by close to 1¼ million consumers of the city of Montreal and adjoining districts is drawn from above the Lachine Rapids. When the Ottawa river flows rapidly it deflects the current of the St. Lawrence, the Ottawa river water, soft and colored, enters the conduit of the Montreal water works, affecting the quality of the supply accordingly.

Although soft water is generally advantageous and economical, the Ottawa river water as seen by the tabulated analyses is rather highly colored and, as it is more expensive to remove color than the usual method of purification at the Montreal water works, the intermixture of Ottawa river water is objectionable.

Thorough mixing of the two waters would take time and need some distance of flow; it is doubtful whether it has been accomplished below the Lachine rapids, as the two samples collected within one hour at St. Lambert and at Longueuil showed on analysis a marked difference in hardness.

Fifty miles below Montreal, at Sorel, the hardness is 93.0 p.p.m. which no doubt represents a uniform mixture, as the difference in the composition of the three samples collected is not appreciable. This is, however, no check on the average composition of the Montreal water supply, because a large part of the Ottawa river water is diverted through the channel north of Montreal.

DISTRIBUTION OF HARD WATERS IN EASTERN CANADA

The computation of results of analyses showing the hardness of the larger public water supplies in Eastern Canada has been summarized in Tables I, II and III, in the hardness map, Figure 2, Figure 1, and in the diagram, Figure 3. Interpreting these data, it must be borne in mind that they relate only to the 53.2 percent of the total population served by the larger water supplies that have been sampled and analyzed in this investigation.

From Table II it is clear that the proportion of persons served by ground water is small, or 5.2 percent of the total population. This is

TABLE I

Number of persons, in thousands, using water of different degrees of hardness from large public supplies in Canada

HARDNESS IN P.P.M.	SURFACE WATER	GROUND WATER	TOTAL
1–10	10.0		10.0
11–20	265.0	8.8	273.8
21–30	218.6		218.6
31–40	93.6	23.6	117.2
41–50	214.0	3.0	217.0
51–60	65.7	5.6	71.3
61–80	31.0	13.2	44.2
81–100	59.2	16.0	75.2
101–120	1335.3	4.0	1339.3
121–180	1050.2	7.0	1057.2
181–250	8.1	60.3	68.4
251–400	65.9	177.6	243.5
401–600		49.4	49.4
	3416.6	368.5	3785.1

TABLE II

Weighted average hardness of water from large public supplies

	SURFACE SUPPLIES			GROUND SUPPLIES			ALL SUPPLIES		
	Average hardness as $CaCO_3$ (parts per million)	Population Served		Average hardness as $CaCO_3$ (parts per million)	Population Served		Average hardness as $CaCO_3$ (parts per million)	Population Served	
		Thousands	Percentage of total population of province		Thousands	Percentage of total population of province		Thousands	Percentage of total population of province
Nova Scotia	24.5	105.8	36.2	124.6	3.1	0.6	26.2	188.9	36.8
New Brunswick	43.7	98.0	24.0	76.8	14.1	3.5	47.9	112.1	27.5
Prince Edward Island				118.3	16.1	18.3	118.3	16.1	18.3
Quebec	90.7	1587.4	55.3	55.0	46.8	1.6	89.7	1634.2	56.9
Ontario*	124.5	1545.4	47.8	302.3	288.4	8.9	152.5	1833.8	56.7
Eastern Canada	102.6	3416.6	48.0	243.0	368.5	5.2	115.8	3785.1	53.2

* Exclusive of territory west of Sault Ste. Marie and northern mining territory.

due to the densely populated districts around the Great Lakes water basin and the more important rivers and lakes.

TABLE III

RANGE OF HARDNESS IN PARTS PER MILLION	POPULATION IN THOUSANDS		
	Surface	Ground	Total
1–60	866.9	41.0	907.9
61–120	1425.5	33.2	1458.7
121–180	1050.2	7.0	1057.2
181+	74.0	287.3	361.3
	3416.6	368.5	3785.1

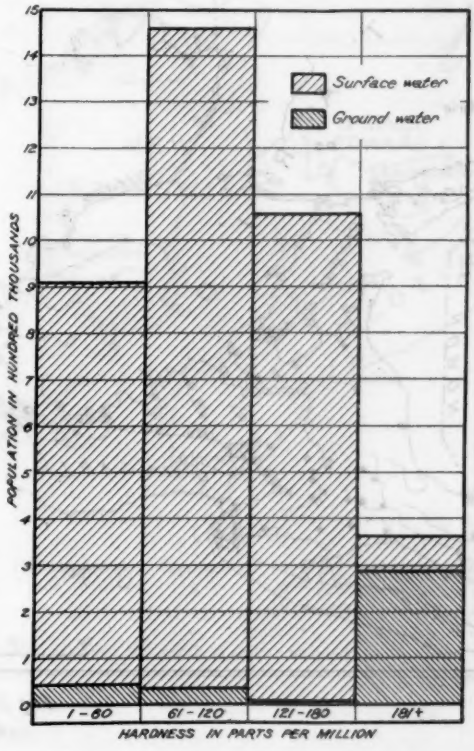


Fig. 3. Diagram showing number of persons using water of certain degrees of hardness from large public supply systems.

Were data available for the total population, the proportion of well water would doubtless be much higher as the rural and smaller community population is usually served by ground water.

As regards hardness, the ground waters, especially in the province of Ontario, are preponderantly hard, the average for the whole of Eastern Canada being classified as very hard, whereas surface waters are medium hard.

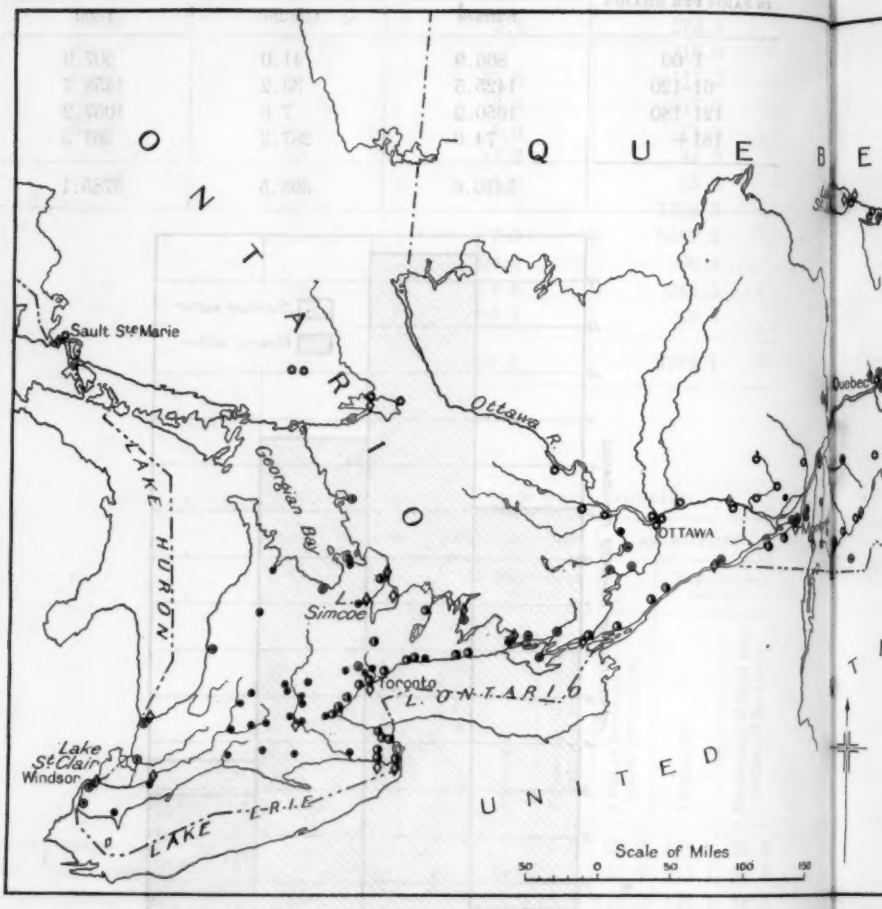


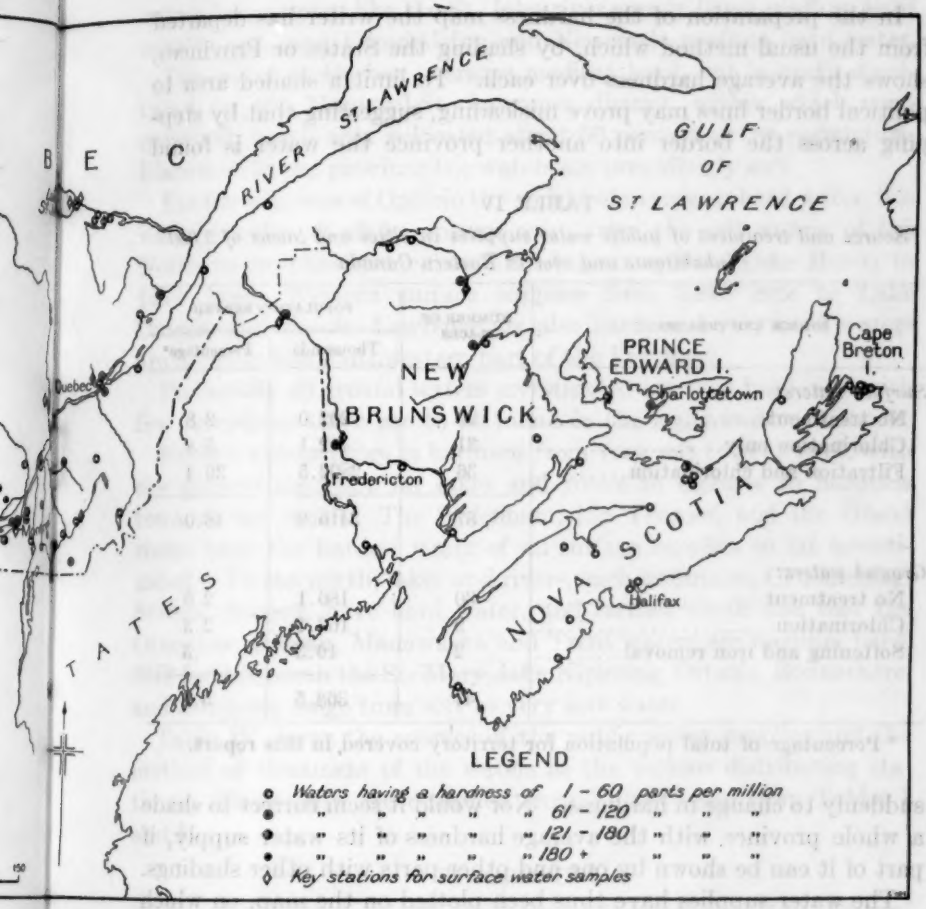
Fig. 2. Map of eastern Can shown the larger civic water ies a

3,785,100 users of water in a territory of 7,115,038 population comprise 53.2 percent of its population and an average hardness of 115.8 p.p.m.

In Table II the hardness data have been summarized from the

analyses for each province, by calculating the weighted average for the hardness of the surface water supplies, of the ground water supplies, and of both under heading "all supplies."

To calculate the weighted average hardness of the surface water


ST LAWRENCE RIVER
GULF OF ST LAWRENCE
PRINCE EDWARD I.
Charlottetown
Cape Breton I.
NEW BRUNSWICK
Fredericton
Quebec
NOVA SCOTIA
Halifax
LEGEND
Waters having a hardness of 1 - 60 parts per million
" " " " " 61 - 120 " " "
" " " " " 121 - 180 " " "
" " " " " 180 + " " "
Key stations for surface water samples

...n Canada showing distribution of ...water lakes according to hardness

supplies, the average hardness of each supply was multiplied by the number of consumers, and the sum of these products was divided by the sum of the number of consumers.

A similar calculation was made for the ground water supplies.

The totals of the products for surface water and ground water were added, and this sum was divided by the total number of consumers to obtain the weighted average hardness of all supplies of a province (2). The results compiled in Table II are graphically represented in the diagram, Figure 3.

In the preparation of the hardness map the writer has departed from the usual method which, by shading the States or Provinces, shows the average hardness over each. To limit a shaded area to political border lines may prove misleading, suggesting that by stepping across the border into another province the water is found

TABLE IV

Source and treatment of public water supplies in cities and towns of 3,000 inhabitants and over in Eastern Canada

SOURCE AND TREATMENT	NUMBER OF PLACES	POPULATION SERVED	
		Thousands	Percentage*
Surface waters:			
No treatment	20	202.0	2.8
Chlorination only	31	412.1	5.8
Filtration and chlorination	36	2802.5	39.4
	87	3416.6	48.0
Ground waters:			
No treatment	30	186.1	2.6
Chlorination	9	163.1	2.3
Softening and iron removal	2	19.3	.3
	41	368.5	5.2

* Percentage of total population for territory covered in this report.

suddenly to change in hardness. Nor would it seem correct to shade a whole province with the average hardness of its water supply, if part of it can be shown by one and other parts with other shadings.

The water supplies have thus been plotted on the map, on which symbols indicate the hardness of the water. The reader, at a glance, can perceive the geographical distribution of the hardness of the various water supplies of Eastern Canada.

In weighted averages the Maritime Provinces are the most uniform, Nova Scotia (with the exception of Pictou, which is hard water) having almost entirely soft water, well within the limit set

for that classification, Yarmouth and Windsor with 6.2 and 7.6 total hardness being the softest waters so far sampled in this investigation.

Prince Edward Island water from its two samples has a hardness range of medium hard, being very near to hard.

The weighted average for New Brunswick is decidedly soft water, only a few supplies like the St. John river supply being medium hard near to soft, and Edmundston and Newcastle medium hard water.

The water of Quebec averages medium hard, but is much dominated by the Montreal-St. Lawrence district, within which comparatively small area is located about 50 percent of the population. Elsewhere in the province the waters are prevailingly soft.

For the province of Ontario the weighted average is hard water, but water supplies of all hardnesses exist, from the soft waters of the Northern supplies to medium hard beginning from Lake Huron to Lake Erie. Western surface supplies from Lake Erie to Lake Ontario and the St. Lawrence are also hard, and very hard waters are found in the southwestern part of the Province.

Practically all ground waters investigated are very hard, and with few exceptions these are to be found in the southwestern districts.

Surface waters range in hardness from very soft to very hard, with the general tendency for lakes and rivers to increase in hardness towards the south. The Sydenham, the Thames, and the Grand rivers have the hardest water of all surface supplies so far investigated. To the north, lakes and rivers, such as Simcoe, Couchiching, Severn, Scugog, have hard water, and farther north and east the Otanabee, Rideau, Madawaska and Trent waters are medium hard. Still farther north the St. Mary, lake Nipissing, Ottawa, Bonnechere, and Sturgeon range from soft to very soft water.

Table IV shows the sources of the public water supplies and the method of treatment of the waters at the various distributing stations. These data are compiled from Table V and from Tables I and II.

REFERENCES

(1) Memorandum Series No. 64, Mines Branch Department of Mines, Ottowa.
(2) U. S. Geological Survey Water Supply Paper, No. 658.
(3) U. S. Geological Survey Professional Paper, No. 135.

NOVA SCOTIA WATER SUPPLIES*

By R. Donald McKay

(*Sanitary Engineer, Dept. of Public Health, Nova Scotia*)

We have at present in Nova Scotia, forty-three water supplies, including three dependent supplies. Of the forty independent systems, two serve cities, thirty-one serve thirty towns, (two of these systems serve separate districts in one town), six serve villages, and one a large institution. Two towns and one village have distribution systems fed from the public supply of adjoining towns. This does not include villages served by a nearby town, but whose system is an integral part of that of the town.

We still have eleven incorporated towns and several large villages without water systems. These towns vary in population from 626 to 1575, and have an average population of 1,055. Topographical difficulties have so far held back several of these towns from building a water works; two of them have within the past three years, had under consideration the alternative proposals of construction of a water system, or a street paving program, and have adopted the latter; two are mining towns whose industrial condition has been poor for some years. There seems relatively little hope that any of these towns will take this step in the near future. This is rather unfortunate, particularly as at least three of them are making a strong bid for popularity as tourist centers.

Of the forty independent supplies, thirty-five are surface supplies (including shallow springs); two are artesian; the remaining three are surface supplies, supplemented by artesian water.

Of the thirty-five surface waters, twenty are lakes; eleven are small ponds, reservoirs on small brooks, or spring-fed reservoirs; two are rivers; one system draws water from both a river and from springs; and one from an impounding reservoir on a small brook, with an auxiliary supply from a river.

Only eight systems treat the water in any way. One lake water is passed through slow sand filters. A second is chlorinated by means

*Presented before the Canadian Section, April, 1937.

of hypochlorite. One institutional plant uses both these safe-guards. Two river waters and three lake waters are treated with liquid chlorine. Two of these latter each have two more or less independent sources; the Halifax system has a high service and a low service, interconnected at a pumping station in the city, but normally fed separately from lakes on different watersheds. The water works of the Town of Yarmouth uses two lakes, in different drainage basins, one as the principal source, the other as an auxiliary.

SUPERVISION

The Department of the Public Health supervises the operation of the various water works. As sanitary engineer for the department, it is my duty to inspect them from time to time. The schedule that I am arranging includes three visits to each. Once in each month, a sample of water from each town on the laboratory list is examined. This list includes every public water system in the Province of which the department is at that time aware; with the exception that during the last two years, two towns have not seen fit to co-operate in this matter, and sample containers sent to them have not been returned to the laboratory.

During the first quarter of 1937, this service has been extended to one village and one institution not previously on the list; and the two towns which have in the past failed to send in samples, have been restored to the list. It may be well to remark that the placing of the institution referred to above on this list does not of necessity mean that the laboratory is undertaking any new work by the move, as samples have very frequently been examined from that source; but it has seemed better to have the samples examined at the same regular interval as those from towns.

Four other town and village systems are expected to take advantage of a routine water examination, beginning next month.

It may be of interest to describe briefly the routine of these examinations. A container is sent each month to some official in the town or village concerned, (in the incorporated towns, they are with two exceptions, sent to the Town Health Officer). In the containers are five test tubes, containing a bile broth, of such volume and strength that when the tube is inoculated with five cc. of water to be tested, the dilution will be that of a standard bile. Also contained in each tube is a standard fermentation vial. The water sample is placed directly in the test tube by the official taking the sample. On receipt

of the tubes at the Laboratory, each tube is inverted, to fill the fermentation vial, and then incubated at 37°C. The tubes are examined for gas at 24 and 48 hours. Cultures showing gas are streaked out on Eosin Methylene Blue agar plates; any typical coli colonies are if necessary, then put through confirmatory tests, and sometimes through tests to differentiate the fecal strain from the non-fecal. In general, however, if the presence of the coliform group is confirmed, we assume it to be of the fecal strain. The differentiating tests are time-consuming, and not too definite; and while our assumption may sometimes give rise to groundless fears, it cannot give us a false sense of security in the presence of danger.

TYPICAL SYSTEMS

Typical Nova Scotia water systems serve towns of about 2500 to 3000 population. Most of these towns are distributing centers for rural districts, many of them with a few industries as well. A few of them depend solely on one industry. About a dozen of them are dependent on the coal industry, and three very largely on the allied industry, steel. Six are built almost completely around the sea fisheries; three others which fell in this class a few years ago, now find the tourist trade of more importance.

The Town of L.... The population of this town was 2727 at the time of the 1931 census. It is a fishing and industrial town. The industries are largely those related to the fisheries, chief among them being ship-building and the manufacture of gas engines.

The source of the water supply is a lake about two and one-half miles from the town, lying in a sparsely settled watershed. Water is pumped from the lake to a one million gallon reservoir, near the boundary of the town, and is distributed from this reservoir by gravity throughout the system.

The distribution system is good. Almost the entire town is well gridded; blow-offs or hydrants are located conveniently; and the mains are flushed thoroughly and periodically. This work is done at night, twice a year with the exception of one long main which does not form a part of the grid, and is consequently flushed more frequently. The flushing is done according to a fixed plan, and each member of the crew knows in advance just what his duties are. Sections are shut off and flushed, one by one. As a result of this careful planning, none of the mains are missed; the sediment stirred up is completely discharged, and not carried into other mains; and

the least possible inconvenience and annoyance is caused the consumer.

It may seem ridiculous to dwell on such a seemingly obvious course of action; but at least in this part of the country, flushing the mains is very often done in the most casual and haphazard fashion. In some towns, the custom is to flush any main only when so many consumers have made complaints, that action can not be longer delayed without the possibility of a riot.

All new hydrants in *L*. . are gated when installed. Most of the older ones have had gates installed from time to time, whenever repairs to the hydrant or hydrant branch have been necessary. This policy is being continued. All hydrants are of a type which shuts off automatically in case of a break, so that even those hydrants which are not yet gated are not serious weak points.

Sanitary features of this system are good. In 1931, a complete survey of the plumbing of the town was made, to check up on possible cross-connections. Since that time, all plumbers have been required to report any changes made. The total number of services now on record is 725, of which 12 are listed as commercial and industrial, the balance as domestic.

The coliform group was not reported from this water during 1935; during 1936, it was reported in five of sixty tubes (five cc. each) tested.

The Town of W. . . . is a coal mining town of about 4000 population. Coal production is its only industry; it is, however, a well established town of many years standing, and more stable than most mining towns.

The water supply is drawn from two sources; a river, and artesian wells; each normally provides about one-half of the total.

Three wells, each eight inches in diameter, and from one hundred and seventy-five to two hundred and sixty feet in depth, are connected to a common gravity main, leading to the distributing reservoir. This reservoir is at an elevation of about 300 feet above sea level (and approximately above river level), and has a capacity of one and one-half million gallons.

Such additional water as is necessary from time to time is pumped from the river to the same reservoir. The pumping station is located on the bank of the river, about a mile from the town, which does not lie in the drainage basin of the water supply. Tile intake pipes lead the water to the pump wells, through a sand bed, which, however,

cannot strictly be termed a filter, though it is so referred to locally. No provision is made for cleaning this sand bed, and it is very doubtful if it serves any useful purpose.

The service pump is a duplex, of one million gallons daily capacity; the stand-by pump is a half-million gallon duplex of a slightly different type. The service boiler is rated at 100 HP, and the stand-by at 50 HP. Both are of the horizontal return tube type, and were manufactured in the province.

Services total 890, of which five are industrial and the balance domestic. Thirty are metered. No estimate is available of the total daily consumption.

The watershed of the river, at the pumping station, has an area of 52.3 square miles, and a population of probably about 500.

The coliform group index for the year 1935, the latest available for that town, was 5 per 100 cc. Other wells have been driven at times, but have yielded small flows; if new wells with approximately the total production of the present ones could be put into service, it is almost certain that the town would have a better water supply than at present.

The Town of I. . . . is another coal mining town, somewhat smaller than the town previously mentioned. It also is dependent solely on this one industry.

The water supply has its source in impounding reservoirs on small brooks; springs also discharge into these reservoirs. The brooks flow through hilly, almost mountainous country, uninhabited and quite heavily wooded.

Water flows by gravity from these ponds to a distribution reservoir near the edge of the town. One of the impounding reservoirs has been out of service for some time. The main from that point to the distribution system was laid largely above ground, and as a consequence, has been so badly cracked as to be useless. As the vulnerability of this main was realized at the time of its construction, arrangements were made to have it de-watered in cold weather, whenever the consumption was small. This, of course, was frequently neglected.

A new dam was built on this brook during the past summer, so that when the line is restored to service, it should provide a considerably larger supply.

The distribution reservoir has a capacity of about eight hundred thousand gallons, or about three days' supply. It is high enough to provide a straight gravity system.

Services number 474, of which 453 are listed as domestic, 20 as commercial, and one as industrial. The latter is the only metered service.

The average coliform group index during the past two years has been 0.7 per 100 cc.

The Town of P. . . . is a shipping and residential town of 1919 people. It was formerly a shipbuilding center of great importance, and considerably larger in population than at present.

The source of the water supply is a brook and spring-fed reservoir about three miles from the town. The capacity of this reservoir is said to be two million gallons. It serves to some extent as a distributing, as well as an impounding, reservoir. The shed is heavily wooded, and is owned by the town.

A second distributing reservoir is located on the opposite side of the town, thus functioning also as a "kick-back" reservoir. This has capacity at normal level of one million gallons.

There are 481 services, 453 being domestic, 27 commercial, and one industrial. None are metered. The average daily consumption is estimated at two hundred thousand gallons.

From the condition of the watershed, the water should usually be good. A slight amount of trouble has been experienced with unauthorized lumbering operations, but no serious difficulties have arisen. The average coliform group index for 1935 and 1936 was 1.5 per 100 cc.

Town of O. . . . This town is the commercial center of a farming district; it is also the home of one or two industries. It is near one of the major coal areas.

The source of the water supply is a group of springs, together with one bored well, in porous sandstone. These feed into a reservoir dug in the country rock, which in turn feeds into a newer, concrete reservoir, into which one or two of the springs are now directly connected. Both reservoirs are fenced. The town owns a large part of the surrounding territory, and the balance is wilderness.

Distribution is by gravity, directly from the reservoirs described.

The population of 1,133 is served by 244 domestic, 20 commercial, and 7 industrial connections.

The bacteriological quality of the water is good. For the past two years, the coliform group index has been 0.7 per 100 cc. Some trouble has been experienced with unsightly growths of algae in the reservoirs. These have not caused any tastes or odors, but have been the cause of some comment in the town, when the reservoirs

have been almost covered with green slime. Considerable trouble has also been experienced with choked mains and services.

The growth seems to start at the bottom, soon extending to the surface, and then spreading rapidly over a large area. The organism responsible was tentatively identified as *Scenedesmus*. Instructions were given the water works superintendent, in the late summer of 1936, dealing with the copper sulphate treatment. Weather conditions, however caused the trouble to clear up at that time, without treatment; it is almost certain that this year, treatment will be necessary. The town has under consideration, the possibility of covering the reservoirs, in order to avoid recurrences of these growths, but will probably not act upon the plan this year.

CROSS-CONNECTIONS: AN INTERESTING PROBLEM

One interesting problem was encountered during the year. One institutional system, mentioned earlier, draws its water supply from a small lake, which is at times subject to considerable pollution. The water is filtered and chlorinated.

The filter is of the English or slow sand type, and was, I believe, designed by Dr. Starkey, of Montreal. While the filter is about thirty-five years old, and the sand in the units, as a result of several replacements, is not now of an ideal grade for its purpose. Still, it has always appeared to be functioning properly. The chlorination is by means of hypochlorite, and while the apparatus is home-made, such installations give good service when carefully operated.

In spite of these safe-guards, however, marked pollution of the house water was frequently reported. The plumbing was of course suspected, but a brief survey revealed no serious defects.

After much questioning and investigation it was found that at the time of the building of the filters, the main which previously supplied the raw water was not disconnected from the pumps by a physical separation, but that a single gate valve had been installed on that line.

At the time, then, that the filters were put into service, the raw water main came from the lake to a point a few feet above the filters, then branched, one line going to the filters; the line from the clear water well ran to the suction side of the pumps. The other branch, (the old raw water main) also went to the pumps; the filtered water line joined it near that point. A single gate valve, a few feet on the lake side of the junction, controlled the cross-connection.

As the contamination of the house water had frequently occurred when the water in the clear well was almost sterile, this cross-connection was strongly indicated as the source of the trouble. The raw water line was immediately ordered cut. The cut was made between the junction and the gate. It was found that there was a small hole through the gate, and that the seat was not in good condition. Water was then flowing through the supposedly closed gate valve, at a rate of perhaps three to five percent of the total water consumption. The actual flow through the valve was about a gallon and a half a minute; the exact total consumption is not known. Moreover, the flow through the valve undoubtedly varied greatly at different times of day.

Since this cross-connection was cut, the coliform group has been consistently absent from the house water samples, and the total bacteria counts much lower. A further confirmation of the opinion that we had found the source of the contamination was that the total count of bacteria in the house water during the period of pollution was roughly what would be expected in a sterile water, inoculated with a proportion of the contaminated lake water, equal to the estimated ratio of flow through the cross-connection, to the total consumption.

I think that this may serve as just one more warning, though it should not be necessary, of the dangers of cross-connections. It might also serve as a warning of the possible presence of unsuspected cross-connections in water systems thought to be in first class order.

WATER WORKS SYSTEMS OF THE MARITIMES*

By J. A. Macmillan

(Manager, Water Works, Charlottetown, P.E.I.)

In the province of Prince Edward Island, which I represent, we have only two publicly owned water systems, one in Charlottetown serving a population of approximately 15,000 people and one in Summerside supplying about 5,000. As the administration in both places is almost identical I shall confine my remarks to the Charlottetown system with which I am more familiar and which is more or less typical of many places in the Maritime Provinces.

The water works system in Charlottetown is municipally owned. It is operated by a commission of three, elected every two years by the voters of the entire city. The election is held at the same time and under the same provisions as that of Mayor. The personnel of the Commission consists usually and traditionally of two prominent business men and an engineer or mechanic. The high type of citizen who stands for office may be judged by the fact that we have had three former Mayors serve as members of the commission. The first Chairman of the Board was the Hon. David Laird at one time a member of the Dominion Cabinet and later Governor of the North West Territories. Once a board is elected and the citizens are satisfied that the members have given good service, they are almost certain to receive an acclamation at the next election and probably for several elections. Our present board, for example, has not had a contest at the polls for ten years or more. Indeed for the past fifty years there have been only sixteen commissioners, or about five complete boards, elected to office. This system takes the management of the water works completely out of ward politics and indeed out of any other kind of politics. It assures continuity of policy and effort and would seem to have a great many advantages over most other forms of water works control. I have personally worked under fourteen of the sixteen commissioners who have served in Charlottetown and I can testify to their sincere desire to give a

*Presented before the Canadian Section, April, 1937.

satisfactory water service to the citizens. The board holds regular monthly meetings, but I have known them to meet daily when circumstances required in much the same manner as the directors of any private corporation. They receive a nominal salary for their services.

The commission operates under an Act of the provincial legislature which gives them fairly wide powers. They are a distinct and separate corporation from the city administration, and apart from paying to the city treasurer the operating surplus of the Department at the end of each quarter, and requesting the council to issue bonds for extensions or permanent improvements, the council in no way interferes with the administration of the water department. All city officials, however, work together in the closest harmony. Any surplus paid to the city by the water department must, according to the provisions of the water works act, be used for water and sewer purposes only,—such as bond interest and sinking fund appropriations and under no circumstances can it be used as ordinary civic revenue. This system may not tend to lower the general civic tax rate but it is pretty good insurance that the holders of water works bonds will receive one hundred cents on the dollar upon the maturity of the bond.

The commission appoints and dismisses its own employees. It employs a Manager, a Mechanical Superintendent, office and other help.

The original system was installed in 1887. The source of supply was then a dug well with a capacity of one million gallons per day. Two steam pumps were installed, which pumped to a reservoir 150 feet above sea level. This supply proved adequate for twelve years or until the introduction of the domestic sewer system made an auxiliary supply necessary. This was obtained about a mile from the original source from driven wells with a capacity of between half and three quarters of a million gallons per day. Oil engines were installed at the auxiliary plant. The administration of the domestic sewage system was placed under the jurisdiction of the water commissioners who have since operated and maintained the sewer system and have been known as Commissioners of Sewers and Water Supply.

With the growth of the town and the natural increase in consumption, it became evident in the late twenties that the existing supply was not going to be sufficient to serve the needs of the community much longer. Consequently in 1930 a new source was located six

miles from the city where a driven-well system capable of supplying an additional one and a half million gallons per day was installed. Four miles of 14-inch force main were laid to the existing reservoir, an electric pumping equipment installed with connections for a diesel standby. As the total supply now greatly exceeds the demand it should not be necessary to seek any further source for many years to come.

Water is now supplied to over 3,000 services who pay either on a flat rate basis or by meter according to requirements. Rates for domestic use are based on the number of fixtures in the homes while all industrial and the larger commercial consumers are metered. Flat rates are collected quarterly in advance while the larger metered services are billed monthly. There is no frontage, rental or any other type of assessment for water works purposes nor is a public hydrant rental collected. Accounts are made out on an addressograph. Domestic bills are delivered by carrier and meter bills are mailed.

There are almost thirty miles of mains (86 percent of which are 6-inch or over) 115 hydrants and 300 gate valves. The services are 99 percent lead. Services up to $\frac{3}{4}$-inch are laid from the main to the street or property line free of charge, but the owner pays all costs from that point to the basement. When a service larger than $\frac{3}{4}$-inch is required, or desired, the property owner pays the difference in cost of street work between $\frac{3}{4}$-inch and the size requested.

The total cost of the water system, both original and subsequent extensions and additions, to date is approximately half a million dollars. The expenditure was financed by bond issues varying in term from fifteen to thirty years. $300,000 of the bonds have been retired and the water works operating surplus is sufficient to take care of all outstanding and future issues of bonds.

We have never experienced much difficulty with frost although the thermometer occasionally drops to 20 below zero. The practice has always been to lay all mains and services well below the frost line, and while it is true that this method of installation makes the initial cost somewhat high this is offset by the low yearly maintenance charges. In fifty years of operation we have never had a frozen main; we have had very few frozen services and less than half a dozen frozen hydrants.

No major extension or new development has ever been made without securing the advice of an outstanding Consulting Engineer. Men like Colonel Waring, M. M. Tidd and Freeman C. Coffin of the

United States, and R. S. and W. S. Lea of Canada have given us invaluable service from the time the original system was designed right up to the present. We have found this practice to be a sound investment.

Charlottetown now has three distinct sources of water supply with more than an ample capacity for our requirements, a duplicate pumping unit at each source, and a concrete covered reservoir with two separate mains to the city. The Provincial Health Department laboratories furnish us with routine analyses of the supply—which is an unusually clear and palatable water. It is not treated in any way and although not one case of illness in fifty years of operation has been traced by the health authorities to the water supply, the matter of treatment is at present receiving serious consideration.

We, in the Maritime Provinces, sometimes receive criticism because of our alleged caution and alleged slow movements. Our civic buildings are often not as imposing as those found in other sections. Indeed even our accounting systems may not be as elaborate as some higher accountants might wish. But may I remind our critics that if they hold any Maritime city's securities they may rest assured on one point,—and it is this,—unless the Atlantic rolls in over us or some equally remote calamity overtakes us they can cash their interest coupons on the due dates and they will receive the face of the bond upon maturity. Our water systems in the Maritimes have never failed to be solvent, or to keep faith with their contracts. That, we think, is a matter for pardonable pride in these misty and uncertain days.

MARITIME WATER SUPPLIES*

By A. A. Laflin

(*Water Works Superintendent, St. Stephen, New Brunswick*)

The Water System takes in three towns on the St. Croix River at the head of tidewater. St. Stephen and Milltown in New Brunswick, and Calais in the State of Maine with a population in the water district between ten and eleven thousand.

The Water System for the three towns on the St. Croix River was first installed and owned by the Maine Water Company, an American company which owned and operated several water works systems in the state of Maine. It was started in 1885 and water turned on in 1886. The pumping station was in Calais, Maine, near the upper bridge. Water was pumped from the river at this point, passed through sand filters, thence to a 3,000,000 gallon open reservoir about a distance of one-half mile on what is known as Todd's Mountain, 180 feet above high tide level. Thence it was distributed to the towns of Calais, Milltown and St. Stephen.

The contract with the town of St. Stephen was for a period of twenty years with the option of renewal or purchase of the distribution system in the town.

About two years before the expiration of the contract, an agitation was started by a few citizens for municipal ownership and a new source of supply—as the river water was not palatable. Later, this was given further impetus by the fact that a new town was going to be built for the manufacture of pulp and paper ten miles up the river at a place called Sprague's Falls, now known as Woodland, Maine, and the waste from which, together with the sewage from that town, would not add to the quality of the water. A plebecite was held at which it was unanimously decided to adopt municipal ownership of the water system. A citizens committee was appointed to act in conjunction with the town council, and at a meeting of this joint committee it was agreed to engage the services of a competent water works engineer. The one engaged was F. A. Barbour of Boston.

* Presented before the Canadian Section, April, 1937.

Following the engineer's study and report, several propositions were submitted to the committee. One was for the Town of St. Stephen only. Another for St. Stephen and Milltown, and a third for St. Stephen, Milltown and Calais. Satisfactory arrangements having been completed with the Maine Water Company for the purchase of the distribution system in the town of St. Stephen, and also for supplying the Maine Water Company with water for the town of Milltown and the city of Calais. The third proposition was adopted.

The work was started early in 1906, completed, and water turned on July 1st, 1907.

The system built to connect with the distribution system purchased from the Maine Water Company consists of a collecting well and a pumping station.

Starting at the pumping station there runs a line of 14-inch cast iron pipe a distance of 4,200 feet until it strikes the main road leading to the town. It is there connected to a 14 x 16 x 16-inch special tee, one branch of which runs 2,000 feet to the reservoir at an elevation 300 feet above high tide level, with a capacity of 1,000,000 gallons. This reservoir is built of cement. The roof is reinforced and supported by 48 cement pillars.

The other branch of the tee is connected to the 16-inch line leading to town. Connecting lines cross underneath the bridge to Calais, Maine and from this branch continuing to the reservoir on Todd's Mountain, which is the property of the Maine Water Company, now known as the Calais Water & Power Company. This reservoir is kept full of water from the St. Stephen well.

The unique feature of this system is that there is a reservoir at each end of the line. If a break occurs in the St. Stephen main line the water is turned on from the reservoir at the other end of the line which in all but two or three cases has been ample supply to take care of the situation until the break was repaired. At the same time the pumping station in Calais is put in readiness to start the pumps in case of fire. If water has to be pumped into the line at any time from the river, it is chlorinated. Of course we get complaints that it does not taste as well as the well water which is almost ice cold at any time of year and needs no treatment.

This is the only system where the source of supply is in one country and distribution mains are in a foreign country.

ABSTRACTS OF WATER WORKS LITERATURE

Key: JOURNAL of the American Water Works Association, **29**: 10 ('37). The figure **29** refers to the volume, 10 to the page of the JOURNAL and ('37) to the year of issue. If volumes of a publication are not paged consecutively but by issues, the figures **29**: 1; 10 ('37) indicate—volume **29**, number **1**, page 10 and 1937 as the year of issue. Initials *W. P. R.* signify that abstract is reproduced, by permission, from *Water Pollution Reports* (*British*).

INDUSTRIAL WATER SUPPLY

Water for Industrial Purposes. W. D. COLLINS. Am. City.

Part I, 52: 7; 52 ('37). The quantities and proportions of the dissolved calcium, magnesium, sodium, bicarbonate, sulfate and chloride largely determine the value of a water for industrial use. Range in mineral matter in U. S. waters is from 20–30 to 500–600 p.p.m. Suitability of water for many uses is determined by hardness, alkalinity and corrosiveness. Hardness is due to the presence of calcium and magnesium salts, and ranges in U. S. public supplies from the 20 p.p.m. of the N. Y. Catskill supply to the 368 p.p.m. of Dayton, Ohio. Softened public supplies are of 80–100 p.p.m. hardness; for industrial purposes, however, hardness may be reduced to any desired degree. Alkalinity generally is measure of bicarbonates of calcium and magnesium, although sometimes sodium bicarbonate is present. Most common type of corrosive water is water with very low conc. of dissolved solids, frequently with high CO_2 content and often with appreciable quantity of organic acids. Other characteristics such as temp., color, suspended matter, or pollution with organic matter may detn. suitability of water for use. *Boiler Use.* Boiler scale is usually formed from hard water but some completely softened waters produce a very hard scale consisting largely of silica. Silica is not removed by zeolite softening and may be present in waters high in carbonate. Early boiler-water treatment was largely internal. This was followed by period with emphasis on external preparation of feed water. At present generally recognized it is practically impossible to prepare water requiring no attention whatever after placing in boiler. Corrosion accelerated by oxygen, carbon dioxide or free hydrochloric acid from decomposition of chlorides of magnesium and calcium is another problem. At high pressures attention must be paid to prevention of caustic embrittlement. Treatment to prevent scale in cooling water or condensers is sometimes necessary. *Air Conditioning.* Character of water available may be a deciding factor in type of air conditioning installed. Temp., presence of precipitating material as iron or suspended matter and corrosiveness are most important factors. *Baking.* Composition of waters has effect on quality of baked goods, investigation of problem led to greater additions of such materials as calcium sulphate, ammonium chloride and potassium bromate than occur in natural waters.

Part II, 52: 8; 55 ('37). *Brewing.* Comparatively few waters are satisfactory for brewing. Because finished product is 85% to 95% water, water used must be tasteless and odorless, free of suspended matter, and iron content vanishingly small. For best results alkalinity must be low and calcium sulfate must sometimes be added to give total hardness of 200 to 300 p.p.m. Sodium chloride is sometimes added in small amounts and pH below 7.0 desirable. *Canneries.* Belief is general that hard water causes hardening of canned peas. Effect of water in canning confined to peas and practically to water of hardness between 200 and 400 p.p.m. *Carbonated Beverages.* For beverage use water should be safe to drink, be free of suspended matter with no dissolved iron content. Keeping quality of carbonated drinks best in waters with alkalinity below 100 p.p.m., under this condition tests indicate freedom of spoilage when kept as long as 3 years. *Confectioners.* Removal of iron in water supply removed one candy manufacturers difficulty in preparing a fondant with the proper color. Tests have indicated that waters with less than 50 or possibly 100 p.p.m., of the usual dissolved mineral constituents would generally be satisfactory in making candy. Usually distilled water with pH of 6.5 to 6.1 favors inversion of sucrose with consequent stickiness which is a detriment to hard candy, but favorable to fondant where inversion is necessary. If distilled water is desirable to use for hard candy, can be corrected by passing through marble to raise pH to as high as 7.4. *Ice Manufacture.* Difficulties with white cores, white deposits and shattering in natural water ice may sometimes be solved by changing amount of air agitation, altering brine temp., and varying practice as to withdrawal of core water. Excessive amount of carbonates may be helped by neutralization with aluminum sulfate or sulfuric acid. If total dissolved solids be expressed as equivalent sulfates, first quality ice can be expected with normal agitation and brine temp. when sulfates are below 171 p.p.m., increase requires more agitation and higher temp., between 342 and 684 p.p.m. production of acceptable ice is doubtful, above 684 good ice cannot be expected. Late commercial zeolites for particular waters are discussed. *Laundering.* Hardness, iron, manganese, rarely color and sediment, are detrimental in laundry work. *Paper Making.* Clear water is about only requirement of water for low-grade paper; iron or color in water may degrade paper quality, and hardness may interfere with sizing. *Textile Plants.* Scouring, resulting in removing of dirt and grease from woolen goods, is interfered with by hard water. Hardness and iron interfere in peroxide bleaching. Water of zero hardness desirable in de-gumming and dyeing of silk. In rayon manufacture requirements are much the same as in treatment of natural fibres.

Part III. 72: 9; 75 ('37). *Available Knowledge of Water Quality.* It is often difficult and sometimes practically impossible to learn the industrial quality of water continuously available at some particular location in U. S. The U. S. Geological Survey with which author is connected has done much work in analyzing and collecting information of U. S. water supplies.—*Martin E. Flentje.*

Water for the Canning Factory. S. LANCEFIELD. Food, 5: 353 ('36). Method is described for determining hardness with soap solution. Clark's

formula being used if results are to be expressed in parts per 100,000 and Wanklyn's if g.p.g. is desired. Methods are also given for free carbon dioxide, dissolved oxygen, silica, calcium, magnesium, iron, manganese, aluminum, sodium and sulfate.—*R. E. Thompson.*

Water for the Canning Factory. S. LANCEFIELD. Food, 5: 387 ('36). Type of water used is important factor in canning of meats, fruits and vegetables. Iron salts cause discoloration of food and temporary hardness salts cause surface scum and cloudiness in liquid portion. For food purposes, it is recommended that water be filtered through sand and softened to less than 5–7 parts per 100,000 but not to zero hardness on account of too great alkalinity. The pH value should not be lower than 7.5. Softening by lime-soda process is recommended. Method is described for determining alkalinity of water.—*R. E. Thompson.*

Water Purification to Meet Paper-Mill Requirements. A. S. BEHRMAN. Paper Trade J., 104: 8; 142 ('37). Discussion of available methods for purifying water to degree required in manufacture of various grades of pulp and paper.—*R. E. Thompson.*

The Significance of Water. HERVEY J. SKINNER. Paper Trade J., 104: 12; 67 ('37). A discussion, from paper making standpoint, of objectionable impurities in water, such as color, suspended matter, iron, manganese and hardness, together with methods for their removal. Necessity of water softening for boiler purposes and sometimes for making up size solution is pointed out and brief comparison of softening by lime-soda and zeolite processes is made. Chlorination for slime prevention and corrective treatment for corrosive waters are also touched upon. Importance of efficient chemical control of treatment is stressed.—*R. E. Thompson.*

Industrial Waters in Canada. Interim Rep. No. 2. H. A. LEVERIN. Bur. of Mines, Canada, Memo. Series, No. 66. 30 pp. ('37). The systematic study of domestic and industrial waters in Canada begun in '34 was continued in '36. For the survey of the hardness of the waters, all communities in Eastern Canada with population of 3,000 or more and in Western Canada with population of 2,000 or more, a total of 235 municipalities, representing 45.5% of the population of the country, were included. The distribution of hardness is extensively discussed and the data obtained are shown, classified in various ways, in a number of tabulations and graphs. In addition, complete analyses of a number of supplies are given.—*R. E. Thompson.*

STREAM POLLUTION AND CONTROL

What Responsibility to Property Owners for Water Pollution? LEO T. PARKER. Munic. San., 8: 462 (Sep. '37). Where citizen discharges deleterious matter into a sewer, city is solely responsible. Citing Johnson v. Kraft-Phenix Cheese Corporation, 94 S. W. (2d) 54, Tennessee, property owner sued to enjoin cheese corporation from discharging refuse matter from its plant in the

city, into a sewer emptying into a creek about 100 yds. above owner's farm. He alleged that the sewage contaminated the water so that livestock will not drink it, rendering his property less valuable. Defendant insisted that liability rested upon city. Court held cheese company not liable. Decision in part: "It is the duty of the city to septicize sewage before deposition in Town Creek." *Damages Allowable.* A city is liable in damages for permanent damages to private property resulting from sewage disposal. Although municipality may be required to expend large sums of money in order to eliminate injuries to private property, this does not justify continuance of injury. Court may refuse injunction against continuation of pollution, but under these circumstances city is liable for reasonable damages. If possible for city to install appliances at reasonable cost court may grant injunction preventing city from continuing to pollute water, and allow property owner temporary damages. If municipality cannot prevent pollution, then court will allow property owner damages reasonably equal to his losses. Under latter circumstances amount of damages may equal full value of the private property. Then municipality may appropriate the property for its own use. Frequently it is impossible to prove positively the source of pollution. Often courts must decide by "inference" who is responsible for pollution. If testimony is convincing, court will allow damages by inference, although no testimony is present which proves positively that the party being sued actually polluted the water. Jackson V. United States Pipe Line Company, 191 Atl. 165, Pennsylvania, reported May 8, '37. A water supply was polluted with oil. Testimony showed defendant's oil pipe line at one point was 80′ from the water source. A 30 yr. old pipe line was never repaired prior to discovery of oil in the water. The line was higher than water source, and intervening ground sloped toward it. Vicinity had no natural oil deposit, nor other oil lines within three miles. Company superintendent notified when oil pollution was discovered. Section of rusty, perforated pipe was replaced. Thereafter oil pollution materially diminished. In view of this testimony, lower court held, and higher court sustained pipe line company liable in damages. *Insufficient Proof.* Courts have held that conjecture is an unsound and unjust foundation for damage verdict. Substantial evidence of facts constituting, not only acts of negligence, but showing that such negligence resulted in damage to the plaintiff, must be introduced, otherwise the latter cannot expect favorable verdict. In Pine v. Bowles, 66 Pac. (2d) 1077, Oklahoma, reported May '37, property owner filed suit against oil company 75 mi. up stream, contending it had contributed to his financial losses resulting from oil and salt discharged into running stream flowing through his land, rendering the water unfit for usual purposes. No evidence was introduced to prove conclusively that pollution complained of actually was same as that discharged by the company into the stream. Although lower court held company liable in damages, higher court reversed the decision. *Law of Navigable Streams.* Is a city liable in damages for discharging sewage into a navigable stream? All courts concede that riparian rights are property; that they are incident to ownership of upland; and that impairment of these rights, by pollution of the stream through sewage discharge, is a taking or a damaging of the property. However, it has been often contended that this rule does

not apply to navigable streams. In view of the later higher court cases it is now well settled that while a city may use a navigable stream for ordinary drainage, this does not include right to mix noxious substances with that drainage in quantities making the dilution inadequate, or their removal impossible without injury to property of others. Such act is in effect an appropriation of bed of river as sewer, and proposition that it may become lawful by reason of necessity is inconsistent with axioms of jurisprudence. Appropriation of the river to carry such substances to the property of another is invasion of his property right. Done for private purpose, it is unjustifiable; for a public purpose, it may become justifiable, but only upon payment of compensation for property thus taken. So, public necessity may justify the taking, but cannot justify taking without compensation. The law is applicable although a state law authorizes municipalities to discharge sewage into navigable streams. Illustration, Parsons v. Sioux Falls, 272 N. W. 288, South Dakota, disclosed that a state law authorizes cities to "empty or discharge sewage or any part thereof into any stream of water within or without limits of the municipality, and for the purpose of the construction and maintenance of such sewer municipality may condemn private property, when necessary, in such manner as is provided by law. . . ." For years city discharged untreated sewage into a river. As volume of sewage increased, undesirable conditions arose. Damage suits were filed against city. Court was requested to prevent further use of river for sewage disposal. Court refused injunction, but allowed property owners damages. Decision given in part. Discharging sewage in tidal waters subject to different rules from ordinary streams. Although authorities not unanimous, it appears in some jurisdictions that authority of the state to discharge sewage into a tidal stream is valid, and property owners have no recourse. See Cityco Realty Co. v. Annapolis, 159 Md. 148; Seaman v. City of New York, 161 N. Y. S. 1002. *Punitive Not Allowable.* Punitive damages are not usually allowable unless pollution was wilful and malicious. In Oil & Refining Company v. Rawlings, 62 Pac. (2d) 1253, Oklahoma, reported '37, suit was filed against oil refining company. Contended that company allowed escape of oil, salt water, etc. into a water supply, and that acts of company in doing so were deliberate, wilful and wanton, or were result of negligence so gross as to constitute wanton and reckless disregard of owner's rights in the water supply, therefore, the company was liable for payment of actual damages, plus punitive damages. Lower court held company liable for $479 actual and $1,000 punitive damages. Higher court held company liable for only actual damages. It also said there was no testimony proving that the water had been polluted wilfully and with malice, although company employees had intentionally and carelessly polluted the water. Any party who diverts or diminishes water thereby effecting losses to owner of private property is liable for actual damages sustained. Illustration, Diehm v. Borough of New Holand, 191 Atl. 393, Pennsylvania, reported May, '37, municipality purchased land about 1½ mi. from certain private property. Municipality built walls around springs and diverted flow, thereby greatly diminishing flow through private property. In summer this diversion left creek practically dry. Remaining water stagnant and unfit for stock. Court held municipality liable. Decision given in part.—*R. E. Noble.*

Judging the Effect of Municipal Sewage on the Receiving Stream by Means of Chemical, Bacteriological and Biological Methods, Using the River Limmat as an Example. E. WASER AND G. BLÖCHLIGER. Mitteilungen aus dem Gebiete der Lebensmitteluntersuchung und Hygiene 28: 3, 120 ('37). Sewage of the city of Zurich after plain sedimentation is emptied into the Limmat. This stream originates in Zurich lake and after a run of approximately 22.6 miles discharges into the Aare. To guard from frequent floods 11.1 mi. of river bed are corrected. Flow varies from 7,926 gal. per sec. at low level to 79,260 gal. during floods, highest run exceeding 184,800 gal. per sec. During cold weather and dry periods in fall flow is very much reduced. Organic load is chiefly derived from sewage from a population equivalent to 235,000, flow averaging 396 gal. per sec.; 77% of settleable solids are removed by 45 minutes sedimentation. Due to a gradient of 0.22% and a resulting velocity of flow of 2.7 ft. per sec. at 13,200 gal., together with low depth and numerous weirs, good aeration and mixing of the Limmat water took place. In 1933 a hydroelectric power station was built at Wettingen including a 61′ dam to store the Limmat water at a distance of 5.6 mi. Storage capacity is 1,637,000 m.g. with approximately 247 acres surface area. In order to investigate variation of Limmat water and of natural purification related to impounding reservoir complete chemical, bacteriological and biological survey extending for over one year before and after filling of reservoir was carried out.

Before storage of water had been begun complete decomposition of organic load at the point where dam was erected was not attained, purity of effluent from Zurich lake not having been nearly attained. Conditions of oxygen saturation were of no sanitary significance as long as the Limmat was a rapid flowing and intensely aerated river. When reservoir was filled, with equal pollution at intake, rapid oxygen consumption took place, conditions in the reservoir being dependent on dilution. During periods of floods with greatest dilution, 1:80 up to 1:120, reasonable purification was observed. With decreasing flow and warmer water in midsummer deoxygenation resulted in water at the dam being considered unsatisfactory. Risk of oxygen depletion was removed by rapidly decreasing water temperature in the fall, rate of oxygen consumption being sufficiently retarded. At all times quality of reservoir effluent was inferior to intake water. In the upper part of the reservoir masses of sludge and filamentous growth accumulated, undergoing acid fermentation. Appearance of reservoir is badly affected by buoyant detritus and use for recreational purposes impossible.—*Manz.*

The Influence of Sulfite Effluents on Rivers and Lakes and the Possibility of Improvements. W. SILFVERSPARRE. Finnish Paper Timber J., '37, 562 (in Swedish) (English summary 587). Sulfur substances in the effluents are likely to show up as hydrogen sulfide when weather conditions are adverse as in Aug. and Sept. Hydrogen sulfide is to be found in bottom water as soon as it contains luxuriant plant growth. If this bottom water is suddenly mixed with surface water having a poor oxygen content, the oxygen will be absorbed and fish-death will occur. In winter, mixing of bottom and surface water in a river is beneficial as at that time the biochemical oxygen demand is low and no hydrogen sulfide develops. Formation of hydrogen sulfide was studied by

using test sticks painted with white lead. Generation of hydrogen sulfide is furthered by presence of cellulose and other organic matter. A number of pictures are given showing sulfur crystals and various organisms as they occur in the water below mills.—*R. E. Thompson.*

The Raritan River Cleanup. C. J. VELZ. Eng. News-Rec., **119**: 717 ('37). Practically all the sewage and industrial wastes in valley of Raritan R. have to date been discharged into river without treatment. As result of order issued by New Jersey State Dept. of Health and with financial assistance of the PWA, 14 projects are expected to be in operation in '38 which will provide for treatment of 30 m.g.d. of sewage and wastes from a contributory population of more than 200,000. Analysis of pollution loads and natural purification capacity of the stream indicated that removal of 65% of oxygen demand of the wastes would be required. Eight of the 13 treatment plants are designed for utilization of coagulants and most of them include chlorination. The State Dept. of Health has established the following minimum standard for effluents discharged into the section of river used for public water supply: suspended solids, 40 p.p.m.; biochemical oxygen demand, 70 p.p.m.; oxygen consumed, 50 p.p.m.; and *Esch. coli* absent in 5-cc. samples.—*R. E. Thompson.*

Cannery Waste Treatment. L. MILLER. Canning Age, **18**: 210 ('37). Screening through No. 40 mesh sieve followed by chemical precipitation afforded sufficient treatment for canneries located on streams providing considerable dilution water. Use of 2 settling tanks is recommended. Addition of 0.5 to 0.75 lb. ferrous sulfate per 1,000 gal. of waste prior to entering primary basin gave good results. Secondary settling is necessary where dilution water is not available in considerable volume.—*R. E. Thompson.*

BOILER FEED WATER

The Deaeration of Boiler Feed Water. R. W. BRUNNER. J. Am. Soc. Naval Engrs., **49**: 5; 127 ('37). Outline of corrosion theory, showing part oxygen plays in depolarization and in increasing surface exposed by roughening due to oxide formation. Due to increased pressures and ratings for Naval boilers with lower weight factors, structures are more vulnerable and therefore require increased attention toward corrosion rate. Reduction in dissolved oxygen by deaeration is considered answer to problem. Various systems of deaerating are discussed and illustrated. Method of testing for dissolved oxygen employed by Navy is modified Winkler test. Attention is being given to spectrophotometer method of testing, which with further development may indicate critical limits. Modified Winkler test is described as requiring extreme precautions in sampling and in basing results on several simultaneous tests. Results are tabulated for various parts of feed water system to indicate variations in oxygen content along line ranging from 0.03 to 1.0 mg. per liter, present aim being to reduce content to 0.10 mg. per liter, maximum. Chemical control of oxygen is not feasible owing to large amounts of chemicals that would be required in ship's stores.—*R. E. Thompson.*

Experiences in Feeding High-Pressure Boilers with Chemically Treated Water. WESLEY AND GEISLER. Chem. Fabrik, '37, 197. Raw water for

100-atmosphere pressure boilers of I. G. Plant at Ludwigshafen is given following treatment: 90% of carbonate precipitated with milk of lime at not over 38°C., precipitate allowed to stand for 2 hours to granulate, excess lime neutralized with aluminum chloride solution, passed through sand filter, then through permutite at not over 38°C., then heated to 95°C., and treated with phosphoric acid and sulfur dioxide. This treatment is described in some detail, with diagram of equipment, analyses of raw, treated and boiler waters, a cut of the boiler and data on composition of steam. Similar data are given for plant at Höchst, where permutite cannot be used on account of temperature. Boilers remain in good condition, corrosion has not been noticed and steam contains about 3 mg. of salts per kg. Turbines foul with soluble salts if boiler water contains 100 mg. sodium hydroxide per liter or over; and the blades, operating at about 250°C., coat with almost pure silica, cause and prevention of which have not been determined. Four references.—*R. E. Thompson.*

Feeding of (Direct Flow) Boilers of the Ramzin System. M. Shkrob. Novosti Tekhniki, 1937: 2; 20. Calcium carbonate, sulfate and sulfite and magnesium hydrate are deposited in zone of preliminary heating of water (economizer), and, therefore, boiler is fed with pure distillate and condensate. Easily soluble salts (sodium chloride, hydroxide, sulfate and carbonate) are deposited in zone of complete evaporation of water and in zone of drying and overheating of vapor. Salts precipitating as hydrates (sodium sulfate and carbonates) are almost completely deposited (80–90%) in boiler in zone of drying and overheating of vapor. Sodium sulfate also promotes precipitation of other salts. Sodium chloride is deposited also in same zone, but percentage of deposition is not over 3–5%, if feeding water has 10–15 mg. sodium chloride per liter. Considerable amount of sodium chloride ("dry") occurs in vapor, causing erosion of turbine vanes. Because of low fusion temperature of sodium hydrate, it separates as a liquid phase, which is carried in form of drops by steam into turbine. Decrease of vapor temperature in turbine causes sodium hydrate deposition on turbine vanes, where it solidifies. To prevent this, it was found that feed water should have dry residue not over 5–7 mg. per liter and pipes of condenser should prevent mixing of raw water with condensate. Chemical washing of boiler should be carried out once or twice each year, using hydrochloric acid in presence of clay, furfural, or other passivators, or using chromic acid.—*R. E. Thompson.*

The Mechanism of Foaming in Boilers. H. Lewis. Power Engr., 30: 21 ('35). Survey of recent articles on causes and prevention of foaming in boilers. Forty-one references.—*R. E. Thompson.*

Water Treatment. L. O. Newton. Chem. Eng. Congr. World Power Conf., 1936, No. L4, 26 pp. Principles of water softening by lime-soda, sodium aluminate and zeolite processes are discussed. Presence of 0.25″ of scale in boiler increases fuel consumption 50%; film of oil 0.5 mm. thick is equivalent in insulating power to scale 5 mm. thick. Oil may be removed from boiler feed water by chemical or electrical coagulation, former with aluminum compounds in conjunction with alkali to raise pH to 9.0 and latter by passing

water through tanks in which electric current is flowing, sodium chloride being added to increase conductivity. Causes of caustic embrittlement are discussed and use of sodium phosphate or hexametaphosphate is recommended for conditioning feed water for use at high pressures. Removal of oxygen, which is also necessary for high-pressure boilers, may be accomplished by spraying feed water into high vacuum at temperature of 60°C.—*R. E. Thompson.*

Electrolytic Treatment of Water for Protection from Corrosion and Boiler Scale, Especially in Hot-Water Systems. T. C. THOMSON. Wärme, 59: 805 ('37). Anode, usually of aluminum, is placed inside boiler, boiler serving as cathode. E. m. f. applied is 6–10 volts. Calcium hydroxide formed precipitates bicarbonate as carbonate. Sulfate hardness is reduced with aluminum or iron anodes but not with carbon or platinum. Current required is very small.—*R. E. Thompson.*

Some Principles of Water Conditioning. P. HAMER. Chem. Eng. Congr. World Power Conf., 1936. Advance Proof No. L5, 24 pp. Fundamental principles of lime-soda process are discussed. Capacity of softening plants may be increased as much as 25% when sodium aluminate is used as coagulant in conjunction with lime-soda treatment. Recent work is described which shows how carbonate-sulfate ratio affects formation and type of boiler scale and how this ratio is affected by presence of sodium salts and changes in boiler pressure. Ratio [chloride in boiler water (blowdown)]/[chloride in feed water] is measure of number times water has been concentrated in boiler. If no sulfate scale is being formed, all of sulfate remains in solution and ratio [sulfate in blowdown] [sulfate in feed] is equal to chloride ratio. If sulfate ratio is less than chloride ratio, then sulfate scale is being deposited; conversely, if greater than chloride ratio, then sulfate scale is going into solution. From these two ratios, amount of sodium carbonate to be added can be determined. Present knowledge of cause of caustic embrittlement is discussed.—*R. E. Thompson.*

Corrosion in Refrigeration Plants. D. W. HAERING. Power Plant Eng., 41: 546 ('37). Abstract of paper presented before N. A. P. R. E. Sodium chromglucosate is recommended as a corrosion inhibitor in brine tanks, condenser systems, boiler feed water and air conditioning apparatus.—*T. E. Larson.*

Chevrolet-Forge High Pressure Plant. W. J. MASSEY AND E. A. SITTER. Power Plant Eng., 41: 516 ('37). The feed water is treated by the lime-soda hot process, with sodium sulfate correction for caustic embrittlement, filtered and deaerated. Colloidal iron is added to remove the last traces of oxygen and to precipitate silica as calcium ferro silicate. Mono sodium phosphate is finally added to prevent scale deposition of the last calcium After six months' operation the scrubbed and super-heated steam has left the main line, valves and turbine blading free from dirt or deposits.—*T. E. Larson.*

Marcus Hook Plant of American Viscose Corporation. H. A. KULJIAN. Combustion, 9: 4; 20 ('37). A description is given of the new 450-lb. power plant which replaces the old low pressure boiler plant and purchased electricity at an estimated saving of $300,000 per year. Zeolite softened Delaware River water is used for the required 25% make up. This with the condensate is deaerated to zero oxygen and treated to maintain a sulfate-carbonate ratio of 3 to 1. pH is maintained between 8 and 9, and the solid concentration in the boilers is held to 2,500 p.p.m.—*T. E. Larson.*

CORROSION CONTROL

The Corrosive Action of Various Types of Water on Household Plumbing. C. P. HOOVER. Ann. Rept. Ohio Conf. Water Purif., 1936, 35. Details are given of extensive tests made in Columbus on corrosion of standard 30-gallon galvanized hot water tanks operated in manner simulating household conditions. The waters used were (1) city tap water, (2) same, passed through zeolite softener, (3) same, plus carbon dioxide, (4) lime-softened, uncarbonated water, (5) city tap water, plus sodium sulfite to remove oxygen, (6) zeolite-softened well water, all at approximately 155°F. At the end of 1 year, tanks were cut open vertically to examine coatings, pittings and tuberculations and photomicrographs were made of sections of the metal. Analyses, including zinc content, of water entering and leaving tanks are given and condition of tanks at end of experiment are described and illustrated by photographs. Water (3) was the least corrosive, notwithstanding fact that pH value was 7.5, while that of others varied from 9.0–10.1 (except 6, which was 7.3). No rust or tubercles appeared in (4) either, but in this case too much protective scale was deposited. Incidentally, the zinc content of this water was highest of all. Order of pitting, from most to least, was 1, 5, 6, 2 and 3–4. Tentative conclusions, the experiments being continued, are: (1) That low-alkalinity water with all or nearly all calcium and magnesium as normal carbonates is more corrosive to galvanized metal than water containing no normal carbonates, even if dissolved oxygen is appreciable and some free carbon dioxide is present. (2) That normal carbonates of calcium and magnesium deposit uneven scale which is somewhat adherent, whereas bicarbonates produce much thinner scale, probably basic carbonate of zinc, which adheres much better. (3) That corrosion of zinc depends more upon alkalinity or pH than upon dissolved oxygen. (4) That the attack of normal carbonate water starts at pin-holes or weak spots in the galvanizing. (5) That products of corrosion produced by bicarbonates fill pin-holes, thus acting as protective film.—*R. E. Thompson.*

Soil Corrosion Studies, 1934—Field Tests of Nonbituminous Coatings for Underground Use. KIRK H. LOGAN AND SCOTT P. EWING. J. Research Natl. Bur. Standards, 18: 361 ('37) (Research Paper No. 982). Observations made on number of metallic and nonmetallic coatings applied to pipe specimens are reported. Specimens calorized by powder process showed somewhat higher rates of loss of weight and somewhat lower rates of maximum penetration than those calorized by dip process. Perforation of lead coatings occurred in most soils. After lead had been punctured, accelerated corrosion took place owing to potential density between lead and steel. Over 10-year period, rates

of loss of weight of galvanized steel were from ½ to ⅓ the rates for bare steel. Galvanized steel corroded most rapidly in poorly drained acid soils and in those high in salts. Coating of 2.8 oz. of zinc per sq. ft. of exposed surface prevented formation of measurable pits in all but one soil for period of 10 years, without appreciable influence of type of ferrous material on which zinc was applied. Rate of corrosion of galvanized steel was lower than that of either steel or zinc alone. Foundry and mill scale had no protective effect. Of nonmetallic materials, vitreous enamel and hard-rubber coatings afforded best protection over 2-year period. Data presented do not permit definite conclusions as to economic value of any of coatings tested.—*R. E. Thompson.*

Engineering Principles Applied to Soil Corrosion Problems. Scott Ewing. Gas, **13**: 5; 23, 106 ('37). Factors concerned in pipe corrosion are reviewed. Equations are given for relation between pit depth and area, pit depth and time and pit depth, area and time. Pipe coating specifications are reviewed with respect to adherence, mechanical characteristics and moisture absorption by coatings.—*R. E. Thompson.*

Protection of Metallic Underground Ducts Against Electrolytic Corrosion. M. Guillot. Rev. gén. élec., **41**: 795 ('37). Causes of electrolytic corrosion by stray currents in ground are explained. Two classes of protection have been developed, those suppressing origin of stray currents and those assuring protection against electrochemical attack by coating, insulation or anodic protection. New method consists in coating metal with conducting envelope which is itself not liable to corrode, e.g., a layer of graphitized rubber. This forms a kind of electronic filter. The various methods are discussed. Nine references.—*R. E. Thompson.*

Economics of Hot Water Conditioning. Anon. Eng. Cont. Rec., **50**: 93; 17 ('37). Brief discussion of prevention of scale and corrosion in hot water systems, with particular reference to use of sodium glucosate, beta glucoside and sulfo glucosate. The most difficult scale or corrosion problems in hot water piping will respond to a properly compounded mixture of these chemicals. Sulfo glucosate combines directly with oxygen to form harmless and desirable oxidation product.—*R. E. Thompson.*

Testing Pipe Coatings. L. H. Louwe Kooijmans. Chem. Weekblad, **33**: 337 ('36). Following tests for porosity of coatings applied to iron pipes are described: (1) "Ferroxyl" test, in which filter paper soaked in aqueous solution of potassium ferricyanide acidified with hydrochloric acid is placed on pipe, blue spots indicating defects in coating. (2) Potassium iodide test, in which muslin soaked in aqueous potassium iodide-starch solution is placed around pipe; lead anode is placed outside this and current passed, pipe forming cathode; iodine is liberated at pores in coating. (3) Hydrochloric acid test, in which quantity of iron passing into solution per unit area on immersion in 0.02 *N* hydrochloric acid for 24 hours is determined. (4)

Conductometric method, in which electrical resistance of coating is determined. 1 and 2 suffer from lack of quantitative definition, while 1 and 3 are vitiated by presence of iron compounds (*e.g.*, soil) on surface of coating. There is a close correlation between results with 2 and 4.—*R. E. Thompson.*

Galvanized Tank Corrosion on Automatic Water Heaters. A. F. CRAVER. Am. Gas J., **5**: 4; 29 and **5**: 27 ('36). Causes and prevention of corrosion in galvanized steel water storage tanks are discussed. Data showing effect of employing various metals for tank fittings are given.—*R. E. Thompson.*

Rôle of the Sulfate-Reducing Bacteria in Metallic Corrosion. H. J. BUNKER. Proc. Brit. Assoc. Refrig., **35**: 1; 152 (1936-37). While intensity of corrosion of iron or steel buried so that it is subjected to anaerobic conditions would ordinarily fall off rapidly owing to polarization of electrolytic action involved in corrosion by hydrogen collecting about iron surface, action of group of vibrio bacteria, reducing naturally occurring [SO_4^{--}] to [S^{--}] and producing much oxygen which unites with polarizing hydrogen, depolarizes corrosion of buried iron and permits it to continue. There is danger of this action wherever the soil contains sulfates and, as the bacteria can survive high concentrations of sulfur and hydrogen, reaction may go on indefinitely. In process, iron is removed from buried pipe, etc., and combines with sulfur to form ferrous sulfide, which is then found in neighborhood of corroded object. Deep layer of graphitized iron is produced on corroded object.—*R. E. Thompson.*

Preparation of Metal Surfaces for Painting. L. A. JORDAN. Chemistry and Industry, 1937, 361. Methods of removing scale, phosphate treatments, metallizing, selection of paints and pigments, and application of paints are reviewed. Twenty-nine references.—*R. E. Thompson.*

Two Unusual Installations of Cathodic Protection of Pipe Lines. GEO. I. RHODES. Proc. Am. Petroleum Inst., Sect. IV, **17**: 21 ('36). Technical and economic data for two installations, one in Louisiana using principle of external power supply, and one in Colorado using zinc-bar anodes containing less than 0.01 percent of lead and cadmium, arc given.—*R. E. Thompson.*

Innovations in the Field of Rust Protection by Painting. M. N. RIVENBURG Drugs, Oils and Paints, **50**: 424 ('35); Chem. Zentr., 1936, I; 2223. Phosphate protective layer is replaced by simple brushing on of liquid containing phosphoric acid, activating salts, binding or wetting agent, etc. Paint coats on iron pretreated in this manner are durable.—*R. E. Thompson.*

Testing Zinc Coating of Galvanized Articles. ANON. Surveyor. **92**: 101 ('37). Specification No. 729-1937, of the British Standards Institution for the testing of zinc coating on galvanized articles other than wire.—*H. E. Babbitt.*

SOFTENING AND IRON REMOVAL

Soft Water And Its Significance For Water Supplies. L. W. HAASE. Die Chemische Fabrik 10: 25/26, 249 ('37). Report refers to recent efforts to promote saving of fats and oils. Statistics show consumption of kitchen soap amounts to 50% of total soap production, laundry soap averaging 30% and approximately 17% in toilet soap. From data furnished by chemical water statistics of German communities only 15% of public water supplies use naturally soft water (up to 89 p.p.m. $CaCo_3$ hardness), derived from districts with archaic noncalcareous formations. Further development of impounding reservoirs being restricted discussion is essentially confined to economics of supplying softened water. Domestic water consumption is estimated at 20% of total amount of water municipally pumped, but a far smaller portion of tap water is considered soap consuming. Little tap water comes in such contact with soap that complete softening reaction may take place. This is demonstrated by the ridiculously large soap consumption per capita which would result if the usual relation between soap loss, volume of water and degree of hardness held true. In order to diminish the waste of soap the soap powder manufacturers recommend presoftening of water. To secure proper dosing of softening reagent and soap powder it is proposed to print the necessary instructions on the packages. For laundries official supervision of process water softening and of use of soda ash seems advisable. From operating data of municipally owned water works of a total of from 660 to 710 mil. gal. of water pumped only 8% exceeds a hardness of 357 p.p.m. In numerous instances mixing of hard and softer water is carried out. Requirements of industrial consumers vary greatly and are not to be considered since it is almost universal practice to maintain individual supplies and installations to meet specific problems. It is never intended completely to soften municipal supplies. Conditions in Germany differ from abroad in that ground water is the general source of drinking water, in preference to the treatment of polluted surface water. Removal of prevailing carbonate hardness with lime incurs lowest expense for chemicals, estimated at one-fifth of present water price for 357 p.p.m. hardness. Softening installations are not readily combined with existing iron removal plants and the like, considerable investment being usually necessary. Use of other chemicals is not advisable from a hygienic viewpoint because of the enrichment of sodium salts, sodium sulfate etc. It does not appear justifiable to presume economic advantages for water softening in every case, or to increase operating costs and water rates as an added burden to small consumers already charged with highest rate. Besides the economic advantages of softening there is to be considered the advantage of the protection of pipe lines, fittings etc. by the ability of the water to build up natural protective coating. This is limited to waters with 36 p.p.m. carbonate hardness. Lowering of pH by split treatment and mixing of soft and hard water and use of CO_2 is mentioned.—*Manz.*

Automatic Water Softening Plant. ANON. The Engr. (Br.) 144: 426 (Oct. 15, '37). Base exchange plant installed by Permutit Co. Ltd. at the West Cheshire Water Board's Poplar Grove pumping station is designed to soften well water with a hardness of 16 deg. (English). The softener units, each with a

capacity of 415,000 g.p.d. (Imp.) of zero water per regeneration, are horizontal steel cylinders, 8′–6″ dia. by 12′–6″ long, designed for internal pressure of 150 lbs. The collecting system, extending beneath the entire zeolite bed, is used to distribute the wash water. It rests on a concrete bed and consists of a manifold pipe, branches, and gun metal strainers fitted with phosphor-bronze screens. The zeolite rests on a layer of crushed flint. A horizontal trough extends above the zeolite bed to distribute the raw water. A separate strainer system distributes the brine solution. The units run continuously, without attention; operation depending on a multiport valve. When the rated output of water has passed, electric contacts in a meter close and initiate the beginning of the reconditioning cycle which is completed automatically. After the first use the brine is regenerated automatically and passed through the softeners a second time. The units are interlocked so that no two may regenerate simultaneously. It is the first plant of its type to be installed by a water works in Europe and the first in the world to embody automatic double salting.—*H. E. Babbitt.*

Removal of Iron from Natural Water. A. SUVOROVA. Novosti Tekhniki, '37, No. 15, 38. Iron was removed by oxidation with chlorine or hydrogen peroxide and by treatment with ferric and aluminum ions with consecutive coagulation of their hydroxides. In presence of humic acid, lowest amount of iron is found at pH 3.0 and 13.0–14.0. In this case, addition of alum caused best precipitation of ferric hydroxide at pH of 3.5–7.5.—*R. E. Thompson.*

STERILIZATION

Straightjacketing Chlorine Tanks When Leaks Develop. ANON. Eng. News-Rec. 119: 100 ('37). Tests were recently made in Chicago to determine effectiveness of immersing ton chlorine cylinders in caustic soda solution to absorb escaping chlorine. Immersion tanks for Cermak Station and Dunne Crib were designed to hold sufficient 20% caustic soda solution to absorb 2,000 lbs. of chlorine, plus 50% excess as factor of safety. Tank, which was constructed of welded steel plates, is 10′–3″ long, 4′–10½″ wide and 6′–6½″ deep. To reduce hazard of spattering, solution is covered with layer of oil, surface of which is 1′–6″ below rim of tank. Tests were made by introducing chlorine through ¾″ pipe, arrangement simulating location of blow-out valve or plug in end of cylinder (3′ submergence). First test was made with all valves wide open, giving maximum flow of gaseous chlorine possible. Maximum rate of 350 lbs. per hour could not be obtained because of frosting, rate dropping to 150 lbs. per hour after 630 lbs. had been absorbed. Not the slightest trace of free chlorine was observed over the surface. Second test was with liquid chlorine, cylinder being allowed to "warm up." Rate reached 700 lbs. per hour within an hour and finally 1,000 lbs. per hour for short time near close of test, 2¼ hours after starting. Average rate of absorption was 600 lbs. per hour. At no time was the slightest trace of chlorine observed above soda solution. Temperatures rose in straight line to 40.5°C., essentially checking earlier laboratory observations of 35–37° at two-thirds saturation. Heavy vibrations were set up: tank, therefore, will be equipped with stiffeners. The ½″ layer of oil was effective in preventing spattering, although by end of test it had completely emulsified to form dense froth.—*R. E. Thompson.*

Chlorination Practice in the Municipal Water Supplies of Ontario. E. W. JOHNSTON. Eng. Cont. Rec. **50**: 68; 26 ('37). In Ontario, there are approximately 225 water supplies, serving 288 municipal systems. Of these, 132 are chlorinated and 64 are also filtered; over 80% of all water distributed is chlorinated. Practice is described and discussed. The Dept. of Health furnishes free of cost all equipment and reagents for conducting *o*-tolidin control tests. Inspections are made and samples examined periodically by the Dept. Typhoid death rate has been reduced from 28.3 per 100,000 in 1910 to 1.1 in 1935.—*R. E. Thompson.*

25 Years of Chlorination of Water Supplies in Germany. HAYO BRUNS. Gas-u. Wasser., Special annual number, **80**: 659 ('37). Chlorination was first proposed by M. Traube in 1894 but did not get practical application until 1907 in the U. S. A. In Germany it was first used in 1912 in the Ruhr district. Chlorine gas was first used at Niagara Falls in 1912. In Germany probably only 15% to 20% of the water pumped is chlorinated, as about one-half of the plants using chlorination apply it only part time. The introduction of the use of chlorine gas was an important step and made chlorination the cheapest method of water purification. Difficulties in chlorination are mentioned,—tastes, phenol chlorine, the influence of organic matter, the formation of chlorhydrates in cold water and the effect of hardness. The tests for chlorine are mentioned, with their limitations, and it is shown that bacteriological tests are needed even with chlorination. The influence of chlorination on the prevalence of typhoid fever is discussed. It is considered that the general reduction of typhoid fever cases since the introduction of chlorination is not as clearly a measure of its effect as are the epidemics occurring at times of temporary interruptions of the chlorination.—*Max Suter.*

Record Size Installation for Chloramine Treatment. ANON. W. Cons. News, **12**: 140 ('37). City of San Francisco installing chloramine plant with capacity of 2000 lb. chlorine and 400 lb. ammonia per day to sterilize Hetch Hetchy water supply. Installed for disinfection and control of growths in tunnels, apparatus is housed in building 28′ x 60′ in plan where room will be available for 15 1-ton chlorine containers and 6 800-lb. ammonia drums. Chlorine is to be fed by 2 solution feed manually controlled vacuum chlorinators with max. cap. of 1000 lb. per day each, min. cap. 60 lb. per day. These machines are arranged for later conversion to fully automatic proportional feeders. Ammonia to be fed by 2 manually controlled solution feed ammoniators each with cap. from 7 to 2000 lb. per day. Details and plans are shown for method of introducing gases to water.—*Martin E. Flentje.*

Colored Scale for Measuring Free Chlorine Dosage from 0.2 Decimilligram to 1 Decimilligram of Chlorine per Liter. LUCIENNE MIGETTE. Rev. hyg. méd. prév., **59**: 238 ('37). A technique, using *o*-tolidine and *p*-nitrophenol, is described.—*R. E. Thompson.*

Electrical Purification of Water. DE LIPKOWSKI. Tech. sanit. munic., **32**: 54 ('37). Ozonization and the operation of ozone equipment are described.

Simplest method of checking ozone dosage is by potassium iodide test in which the iodide is decomposed by ozone quantitatively in presence of sulfuric acid.—*R. E. Thompson.*

HEALTH AND HYGIENE

Endemic Fluorosis in the Nellore District of South India. H. E. SHORTT, C. G. PANDIT AND T. N. S. RAGHAVACHARI. Indian Med. Gazette, 72: 396 ('37). (Reprint). A brief preliminary report on prevalence of endemic fluorosis in Nellore district, north of Madras. Early manifestations of disease in children is chocolate-colored mottling or banding of teeth or dull but pure chalky-white appearance. No other effects are noticeable in children. These symptoms were observed in over 50% of school children in the several villages so far investigated. Later effects, in adults of 30 years and upwards, include pain in spinal column, especially in cervical and lumbo-sacral regions, and some stiffness and pain in joints of both upper and lower extremities; there is usually anorexia. Final stages of disease, generally evident in people 40 years of age or older, are summarized as follows: (1) Complete rigidity of spine including cervical region so that affected person to turn head must turn the whole body. (2) Rigidity of joints of both upper and lower limbs with inability to assume squatting position. (3) Fixation of thoracic walls so that breathing becomes entirely diaphragmatic, while chest is often barrel-shaped in an antero-posterior view but flattened in front. (4) Symptoms of pressure on spine may be present without actual spinal deformity. (5) Patient is eventually completely bed-ridden and usually dies of some inter-current infection. A brief outline is given of geology and physiography of the district, in which are located mica mines, a mineral often found associated with deposits containing fluorine compounds. Fluorides were found in 19 of some 60 specimens of rock taken from well diggings, river and tank beds and mines and mine areas. Chief source of drinking water of villages is shallow wells. Preliminary field tests on water of these wells showed fluorine contents of 0–10 p.p.m., high fluorine content often being associated with high total solids. As a general rule, incidence of disease was in direct proportion to fluorine content of water supply. Securing of water from greater depths may possibly be a solution of the problem. Treatment of water for removal of fluorine would probably be too costly for practical application.—*R. E. Thompson.*

Chronic Endemic Dental Fluorosis in Ohio. A. L. VAN HORN. Ann. Rept. Ohio Conf. Water Purif. 1936, 21. More than 400 water supplies in Ohio have been examined and 48 have been found to contain more than 1.0 p.p.m. fluorine, i.e., the minimal threshold concentration that studies have indicated will produce mottled tooth enamel. Maximum fluorine content found was 4.0 p.p.m. Wells producing water of high fluorine content are located chiefly in northwest section of state, many of them penetrating the Niagara limestone. There appears to be no definite correlation between depth of well and fluorine content, which varies widely in wells in same community. Detailed description of various degrees of mottling are given, together with results of mottled enamel surveys in 5 communities in which fluorine content of water supply varied from 1.4 to 2.6 p.p.m. Percentage of school children (9–14 years of

age) affected who had been continuous residents varied from 20 to 72, mottling being mild in most cases. Incidence of gingivitis and other mouth infections was higher in affected than in non-affected groups. Recent investigations have indicated that the entire skeletal system may be affected. Eleven references.—*R. E. Thompson.*

Milk as a Preventive of Mottled Enamel of the Teeth. JAMES A. TOBEY. Milk Plant Monthly, **26**: 1; 30 ('37). Presence of fluorine in drinking water and absence of adequate content of calcium in diet are the factors concerned with mottled tooth enamel. Fluorine, to extent of 2–5 p.p.m., in drinking water will cause defective enamel in children's teeth. Recommended that where such waters exist milk be substituted for it; this will also supply adequate calcium. Fluorine content of milk is very low even when cattle receive substantial amounts in their drinking water.—*R. E. Thompson.*

Lead Poisoning by Drinking Water. The Influence of Deacidification and Remineralization on the Solubility of Lead in Pipes. A. PELLÉ AND ARTUS. Bull. acad. méd., 117: 156 ('37). Solubility of lead in water is lowest at pH 7.6–7.8. If water is softened to low mineral content, remineralization with calcium sulfate and calcium carbonate + carbon dioxide is necessary to prevent absorption of lead. Plating with tin gives good protection.—*R. E. Thompson.*

Public Indoor Swimming Pools of Connecticut. J. H. L. GILES. Conn. Health Bulletin, **51**: 101 ('37). A survey of 50 Connecticut indoor swimming pools has recently been completed by the State Department of Health. Field tests for residual chlorine and pH of the water in each pool were made, and samples were collected from the water at each pool for bacteriological examination, a tabulation of results being given.—*G. C. Houser.*

Safe Roadside Water Supplies in Ohio. F. D. STEWART. Civ. Eng. **7**: 675 ('37). Rigid requirements have been established by the State Department of Health for roadside wells bearing a seal of safety. Samples collected from the wells are analyzed in the State laboratories. In locating wells thought is given to attractiveness of location and to minimize traffic hazards. The wells are maintained by the Highway Department. It is also expected to provide similar wells in state roadside parks. All the construction is to be done by the personnel of the State Highway Department.—*H. E. Babbitt.*

CHEMICAL

Removal of Fluorides from Water. ELIAS ELVOVE. U. S. Pub. Hlth. Repts. **52**: 1308 (Sep. 17, '37). Tricalcium phosphate, magnesium oxide, and magnesium hydroxide appeared promising in consideration of materials available for removal of fluorides from water. In exptl. work reported on in paper, most tests were made with magnesium oxide because this material is commercially available and not expensive. Two grades used, (1) calcined magnesite as used in cement floors, and (2) light magnesium oxide similar to U. S. P. grade; both finely divided. In tests 1 oz. per gal. was added to dis-

tilled water containing 5 p.p.m. fluoride (F). This was well agitated with air and allowed to settle until clear, usually overnight. Same material used over a number of times to detn. how much fluoride a given quantity would remove. On first tests original dose of material was found to reduce fluoride content from 5 p.p.m. to 1 p.p.m. or less when used in 3 runs for the calcined magnesite and for 4 runs with the light magnesium oxide; in the first case the composite sample of the first 4 runs still had content less than 1 p.p.m. (0.8) and with second material composite of first seven runs was within this limit. By using both materials a second time further removal was secured. On basis of double treatment author calculates 1 lb. of calcined magnesite would serve to treat about 88 gal. of water and 1 lb. of the light magnesium oxide about 96 gal. bringing 5 p.p.m. fluoride down to 1 p.p.m. Suggestion is made that magnesium compds. so used might have resale value for other uses. 9 tables and bibliography complete paper.—*Martin E. Flentje.*

Use of a Photocell for the Colorimetric Determination of Fluorides in Drinking Water. V. P. Shvedov. J. Applied Chem. (U. S. S. R.), **10**: 940 (in German 945) ('37). The photocell "selen-3" (described) was successfully used in determination of fluorides in water. The I. H. de Boer and Steiger-Merwin methods gave best results. Six references.—*R. E. Thompson.*

Determination of Minute Quantities of Lead. Percy G. Jackson. J. Soc. Chem. Ind., **56**: 211–13T ('37). Tests were made to determine effect of varying quantities of acetic acid, potassium cyanide, copper, iron and gelatin, and the order of introducing reagents in colorimetric determination of lead as sulfide. Depth of color was found greater in presence of acetic acid than in ammonia and in 50 cc. of solution tested in Nessler tube it is well to have 2 cc. of 33% acetic acid present. Presence of as little as 0.001 mg. Cu reduces color of 0.1 mg. of lead. Ferric iron also affects color similarly but ferrous iron has little effect. Potassium cyanide does not counteract effect of copper but in ammoniacal solutions causes even greater diminution of color. It is necessary to remove copper and ferric iron before making test. Varying proportions of potassium cyanide affect depth of color and gelatin reduces color but its presence is desirable when not less than 0.02 mg. of lead is present, as colors undergo less change on standing. Hydrogen sulfide solution must be fresh and saturated with gas. Following method for determining lead in beer is given: Take 50 cc. sample and oxidize with nitric acid, with only 0.5 cc. of sulfuric acid added toward the end. Evaporate off most of latter, extract residue with 12 drops of dilute nitric acid + 5 cc. of water. Heat, filter through paper and wash with a little hot water. Cool, neutralize to methyl orange with ammonium hydroxide and leave solution a slight pink. Saturate with hydrogen sulfide and compare color with standards. Attempts were made to precipitate lead sulfate, dissolve precipitate in ammonium acetate and determine lead colorimetrically but there were slight losses of lead unless very little sulfuric acid was used. This method is preferable when more than 0.5 mg. of lead is present.—*R. E. Thompson.*

Photoelectric Determination of Phosphorus in Estuarine Waters. C. E. Brambel and R. P. Cowles. Science, **85**: 340 ('37). Wide variations in

salinity and heavy metal content interfere with determination of phosphorus in estuarine waters by Denige's method. A photoelectric colorimeter, therefore, was devised to study color production by the reduced phosphomolybdate which is measured in Denige's method by visual comparison with a standard blue. A Weston photometer measuring light intensity in foot candles, a ruby glass filter, a 30 cm. Pyrex tube closed at both ends to hold the solution and a Spencer microscope served as the photoelectric colorimeter. Calibration curves were plotted for salinities ranging from 0 to 3% with 0.5% increments. Artificial sea water and known amounts of potassium dihydrogen phosphate were used in preparing the curves. Contrary to expectations, copper and iron in quantities reaching 4–5 times actual amount likely to be present had only negligible effects on color produced. Interference of silica was prevented by using molybdate reagent of high sulfuric acid content. Results show that there is decided salt effect up to 0.5% sodium chloride but from this quantity to 3% there is very little change.—*R. E. Thompson.*

A Method for Determination of Phosphate in Water. H. LIANDER. Iva, '37, 103; cf. Ann. Chem. Pharm., **115**: 248 (1866); J. Pharm. Chem., **26**: 219 (1854). Dissolve 40 grams ammonium molybdate in 100 cc. of warm water, filter, add 15 cc. of 1% aqueous solution of quinine sulfate and 100 cc. of concentrated sulfuric acid in 75 cc. of water. Add 1 cc. of this reagent to 10 cc. of water sample and observe turbidity in comparator. Silica interferes, necessitating application of a correction.—*R. E. Thompson.*

Detection and Determination of Ammonia in Waters. N. D. COSTEANU. J. pharm. chim., **25**: 101 ('37). Distill the water to be analyzed with sodium hydroxide, soak strip of filter paper in distillate, and allow water adhering to paper to evaporate spontaneously. Compare color produced on this paper by 1 drop of Nessler solution with tints similarly obtained after dipping papers into solutions of pure ammonium chloride in water, corresponding to, respectively, e.g., 1, 0.5, 0.1, 0.05 and 0.01 mg. ammonia per cc. Prepare Nessler solution with only half usual quantity of sodium hydroxide to avoid crimping the paper. Method is said to be sensitive to nearly 2×10^{-6} mg.—*R. E. Thompson.*

Liquid Film in Gas Absorption. MARGARET H. HUTCHINSON AND T. K. SHERWOOD. Ind. Eng. Chem., **29**: 836 ('37). Absorption rates in water of eight pure gases (including oxygen, carbon dioxide, sulfur dioxide and chlorine) were determined in investigating effect of solute gas on absorption coefficient under conditions where liquid film resistance controls. Runs were made at stirrer speeds of 0 to 1025 r.p.m. with gas exposed above stirrer water surface. Liquid film coefficient is concluded to be substantially independent of such properties of solute gas as molecular weight, diffusivity and solubility in water, at high stirrer speeds. At low speeds and at rest, coefficients were influenced by natural convection effects and were independent of stirrer speed. —*Selma Gottlieb.*

Critical Discussion of Previous Inhibitor Theories and Explanation of the Function of Organic Inhibitors in the Acid Corrosion of Metals. W. MACHU.

Korrosion u. Metallschutz, 13: 20 ('37). None of heretofore proposed inhibitor theories can explain all phenomena in satisfactory manner. Presence of more than 50% pores in area of adsorption layer is against assumption of a purely protective layer. Overvoltage theory is contradicted by many experimental observations. Changes in surface tension and viscosity due to added inhibitor are too insignificant to explain powerful action and there are strong arguments against assumption of formation of complex compounds between H ions and inhibitor groups. Author's theory is based on assumption that a very porous layer is formed by pure adsorption, which has high electrical resistivity due to narrowness of channels between molecular chains. Thus rate of diffusion and ion transfer in metal-solution interface layer is greatly reduced. Thirty-seven references.—*R. E. Thompson.*

Colloidal Properties of Hydrous Alumina. THURSTON E. LARSON. Chem. Age (London), 36: 439 ('37). Experiments indicated that a solution of sodium hydroxide and aluminum chloride hydrolyzed to give positively charged particles when initial pH was above 8.2, and negatively charged particles when pH was below 8.2. This checks work of Mattson. Isoelectric point is broadened to pH 7.5 when sulfate is present. Aging inhibits adsorptive power of colloidal hydrous alumina. When sodium aluminate was used in conjunction with lime and soda ash, residual hardness was always lower than when lime or lime and soda alone was used. The OH ion was more effective in precipitating magnesium than was the aluminate ion.—*R. E. Thompson.*

Persistence of Fluorescein in Soils, Etc. Influence of Ferruginous Formations. ALEXANDRE GUILLERD AND PIERRE ÉTRILLARD. Compt. rend., 205: 77 ('37). Waters of certain spring still showed traces of fluorscein, which could only have been placed in source of water long previously—in the case of spring originating in Lower Eocene limestone of Paris basin, 54 months. Accordingly, fragments of limestone of that formation were soaked in solution (300 grams/23 liters) of fluorescein (uranin) for 24 hours and then repeatedly automatically washed with pure rain water. They still showed quite distinct traces of fluorescein after 420 days; amount extracted was greatly increased if wash water was slightly acid (pH 5.2). Filtering a solution containing fluorescein through sand to which 5% colcothar had been added in no way diminished content of fluorescein; nor was this affected when the ferric hydroxide separated by flocculation from ferruginous water to which dye had been added.—*R. E. Thompson.*

LABORATORY METHODS

Sources of Error in the Identification of Ammonia in Drinking Water by Means of the Nessler Reagent. A. REUSS. Z. Untersuch. Lebensm., 73: 50 ('37). It is pointed out that rubber stoppers can be attacked by Nessler reagent producing results simulating presence of ammonia in water. If there is a material amount of iron in water, colloidal ferrous hydrate may be formed which is rapidly changed to ferric hydrate, imparting to water a yellowish tint, thus giving deceptive results. Seignette salt, often recommended to prevent precipitation of alkaline earths, can also give rise to misleading results. It has often been observed that water containing small amount of

ammonia gave no color in presence of Seignette salt, but did so when this salt was not used and potassium or sodium carbonate had been added. According to author's experience, potassium and sodium hydrates always react with Nessler reagent because of presence of traces of ammonia; these traces may be removed by heating alkali in twice its weight of water for at least 15 min. For precipitation of alkaline earths, author uses mixture of equal volumes of 10% potassium hydrate and 33% sodium carbonate (Na_2CO_3. $10H_2O$). Lack of OH^- ions prevents start of reaction with Nessler reagent; it can take place only in strongly alkaline solutions. Great excess of ammonium salts also prevents onset of reaction. Very small amounts of hydrogen sulfide can also simulate start of reaction with Nessler reagent. Larger amounts of hydrogen are not to be overlooked and can be rendered harmless by use of zinc or cadmium acetate. Many samples of water have property of converting ammonia rapidly through oxidation to nitrite. It has frequently been observed in parallel tests of same water that ammonia alone may be found in some and only nitrite in others. It is necessary in parallel examinations to test for both in each sample.—*R. E. Thompson.*

A New Fluorine Reagent. C. F. MILLER. Chemist-Analyst, **26**: 35 ('37). Dissolve 1.84 grams of benzidine in glacial acetic acid, dilute to 500 cc. and mix with 500 cc. of 0.02 N mercury succinimide solution. To make test, neutralize unknown solution with sodium hydroxide, make just acid with acetic acid, heat to 50°C. and add excess of reagent. As little as 0.04 mg. in 10 cc. will give precipitate of $Bz_2(HF)_4$. HgF_2 (where Bz is benzidine molecule) if oxidizing agents, sulfuric and phosphoric acids are absent. Precipitate can be washed with cold water, dried over sulfuric acid and weighed.—*R. E. Thompson.*

Microchemical Examination of Drinking Water. J. GANGL AND A. WASCHKAU. Mikrochemie, **22**: 78 ('37). Usual methods of complete examination of potable water require about 2 liters of water; for microchemical examination, only about 200 cc. is necessary. In present paper, detailed directions are given for determining residue on evaporation of 5 cc. of water. Results obtained after drying at various temperatures and testing by adding known quantities of different salts of calcium, magnesium, and sodium to pure water were compared with those obtained with 200 cc. of water. Conclusion is drawn that results obtained by evaporating 5 cc. of water to dryness in platinum dish, drying at 180° for an hour and then in dessicator over fused calcium chloride are so favorable that method deserves adoption in Austrian Pharmacopoeia.—*R. E. Thompson.*

Determination of Temporary Hardness of Water. V. T. CHUĬKO. Zavodskaya Lab., **5**: 1422 ('36). Examination of determination of temporary hardness of water by boiling for definite periods of time showed that decomposition of calcium and magnesium bicarbonates in natural waters proceeds only in intermediate stage according to equation of reaction of second order. Amount of residual alkalinity formed in determinations is influenced by presence of organic matter in the water, extent of surface contact between water and

vessel walls, presence of catalysts in reaction sphere and correlation between equivalents of calcium, magnesium and bicarbonate ions. Hence, determination of temporary hardness gives no indication of actual boiler scale-forming properties of water. This can only be obtained from determination of carbonate hardness.—*R. E. Thompson.*

Colorimetric Method for the Determination of Traces of Phenol in Water. G. U. HOUGHTON AND R. G. PELLY. Analyst, 62: 117 ('37). The phenol is converted into indophenol by well-known method of oxidation with sodium hypochlorite in presence of dimethyl-*p*-phenylenediamine. Resulting indophenols are easily dissolved by carbon tetrachloride so that sensitivity of method can be increased, when phenol content is less than 0.15 p.p.m., by extracting with carbon tetrachloride and matching color of extract.—*R. E. Thompson.*

Water Analysis. F. T. JEWSON. Chemistry and Industry, '36, 983. Hehner's method for determination of calcium and magnesium, modified by neutralizing before addition of equal volumes of 0.1 molar sodium hydroxide and sodium carbonate, gives total calcium + magnesium; omitting the sodium carbonate and removing the magnesium hydrate gives the magnesium, the calcium being found by difference.—*R. E. Thompson.*

Determination of Silver in Wine or Other Fluids Treated by the Katadyn Process. ALADÁR JENDRASSIK AND SZILÁRD PAPP. Kisérletugyi Közlemények, 39: 207 ('36). The small quantity of silver in these liquids is mostly in form of organic complex. Liquid, therefore, should be concentrated by evaporation, the organic matter destroyed by digestion with sulfuric and nitric acids, excess of latter removed by heating and resulting silver sulfate solution diluted, neutralized with ammonium hydroxide and silver determined colorimetrically by means of solution of *p*-dimethylaminobenzorhodanine in acetone. —*R. E. Thompson.*

Water Analysis. Volumetric Determination of Alkalinity, Hardness, Calcium, Magnesium, Sulfates, Chlorine and Carbon Dioxide. G. CHRISTEN AND G. BERRAZ. Anales asoc. quím. argentina, 24: 155 ('36). Following analytical scheme is adaptation of volumetric methods to determine main analytical data of water. For determination of calcium and magnesium, simultaneous determination of both cations is used. If, to exactly neutral solution containing both calcium and magnesium known quantities of sodium carbonate and sodium hydroxide solutions are added, first precipitates quantitatively as calcium carbonate and second precipitates as magnesium hydroxide. Separating precipitate by filtering and determining excess of each reagent by alkalimetry give calcium and magnesium contents. The two end points in evaluation of mixture of sodium carbonate and hydroxide are determined previously by potentiometry, but these are compared with use of suitable indicators which could attain same result. For first end point, mixed indicator of thymol blue and cresol red is used; for second, bromocresol green. All operations were in absence of carbon dioxide. Alkalinity of water is deter-

mined with 0.1 N HCl, using bromocresol green as indicator. For chloride determination, Votocek-Kolthoff method is preferred to that of Mohr. It is based on very low degree of dissociation of mercuric chloride, formed by addition of known mercuric nitrate solution to water acidified with nitric acid. End point is recognized much more clearly by use of sodium nitroprussiate indicator, which gives turbidity in presence of slight excess of mercury. Sulfate is determined by benzidine method. Carbon dioxide is determined with 0.1 N sodium hydroxide, free from carbonate, with thymol blue/cresol red indicator.—*R. E. Thompson.*

Graphic Aids for Making Water Purification Calculations. O. F. Schoepfle. Ann. Rept. Ohio Conf. Water Purif., '36, 58. Two examples are given: (1) The construction, by non-mathematical process, of nomograph for solving the formula: m.g. × C/f.h. = m.g.a.d., where m.g. is mil. gal., C a constant peculiar to each plant, f.h. the number of filter hours and m.g.a.d. is mil. gal. per acre per day. (2) Pearson's Square method of computing dilutions, e.g., amount of solvent or solution of lesser strength required to reduce concentration of known solution to predetermined value.—*R. E. Thompson.*

Dithizone, an Excellent Reagent for Lead. C. Franklin Miller. Chemist Analyst, **26**: 55 ('37). Preparation of the reagent from phenylhydrazine is described, together with procedure for testing for lead with 0.01% solution of reagent in carbon tetrachloride.—*R. E. Thompson.*

Method for the Determination of Ozone at Very Low Concentrations, Based on the Catalytic Effect upon the Oxidation of an Aldehyde. E. Briner and E. Perrottet. Helv. Chim. Acta, **20**: 293 ('37) (in French). Ozone at concentrations of 10^{-8} to 10^{-6} can be determined satisfactorily by passing gas containing it into solution of benzaldehyde in carbon tetrachloride. Aldehyde is oxidized to acid which can be titrated with alkali hydroxide.—*R. E. Thompson.*

Colorimetric Determination of Aluminum with Alizarin. A. K. Babko. Univ. état Kiev, Bull. sci., Recueil chim., **2**: 2; 59 ('36). Chief conditions necessary for colorimetric determination of aluminum with alizarin are the proper concentration of alizarin and its ratio to aluminum. In concentrated solutions, alizarinate is present in form of adsorption compound of varying composition. Composition depends upon pH of solution. Conditions necessary for obtaining colored compounds were investigated and method for determining alimunum alizarinate in aqueous solutions was developed. To solution containing 0.015–0.08 mg. aluminum add 0.75 cc. of 0.001 *M* alizarin (sulfonic acid salt), then add 1 cc. *N* sodium acetate, and after 3–5 min. acidify with 1 cc. of normal acetic acid and dilute to 10 cc. Make colorimetric comparison with solution containing free alizarin and with solution of alizarin containing excess of aluminum (the lake form). Amount of lake is first found, and from this, aluminum is determined.—*R. E. Thompson.*

"Blacher Method" for Determining Hardness in Waters. F. C. Saville. J. Soc. Chem. Ind., **55**: 346T ('36). Method is discussed particularly with

reference to determination of magnesium after removal of calcium with sodium oxalate. Filtration to remove calcium oxalate is recommended. Results are given of titrations of 0.2 N calcium chloride and 0.02 N magnesium sulfate, separately and mixed, with 0.1 N potassium palmitate, using *o*-cresolphthalein as indicator. It is concluded that method is accurate for determination of total hardness but not for determination of magnesium hardness after removing calcium as oxalate.—*R. E. Thompson.*

Accuracy of Boron Determinations Carried Out in Pyrex Glassware. R. LAGRANGE. Documentation sci., **5**: 108 ('36); chimie industrie, **37**: 869. In determination of boron in mineral waters by distillation of the methylboric ester, it is impossible to use Bohemian glass flasks, which are too fragile, or silica flasks, which are too expensive. Jena glass and Pyrex have the disadvantage of containing boron. It is therefore necessary to take this fact into consideration and apply correction, which, in some cases, may be as much as 5-10% of amount found.—*R. E. Thompson.*

Measurement of the Air Content of Water. FUKUSABURO NUMACHI. Sci. Repts. Tôhoku Imp. Univ., 1st Ser., K. Honda Anniversary Vol., '36, 501 (in German). By suitable manipulation of leveling tube and series of stopcocks, 100 cc. of water is introduced into apparatus and air is removed from it at room temperature by producing a vacuum. About 1 hour is required to remove all air. Series of experiments show that most natural waters are very nearly saturated with air. Water saturated with air at 15°C., contains 0.0209 cc. air in 1 cc. water.—*R. E. Thompson.*

Some Laboratory Kinks and Time Savers. C. A. HABERMEHL. W. Wks. and Sew., **84**: 3; 96 ('37). Description of photo-electric turbidimeter, titration table, automatic suspended solids filtration and weighing, and special pipette rack.—*H. E. Hudson, Jr.*

WATER SUPPLY—GENERAL

Pasadena, Calif., Water Dept., Annual Report, '36-'37. 73 pp. MORRIS S. JONES, Chief Eng. and Genl. Mgr. to C. W. Koiner, City Mgr. covering fiscal year ending June 30, '37. Annual income from water sales, permits and changes, $870,263.84, an increase of 0.52% over previous year; profit for year $62,632.11, including non-operating revenues of $48,427.90. Operating expense including maintenance, depreciation, bond interest and redemption increased 15.6% to $856,059.60, however actual increase in operating expense was only $5,013.93. Total cost of water produced equalled 15.51¢ per 100 cu. ft., water production 607,304,200 cu. ft. with 551,805,700 cu. ft. sold. Rainfall for year amounted to 142% of normal with 28.81 in., 55 yr. mean being 20.24 in., max. temp. for yr. was 104, min. 23, mean 63.1, 29 yr. mean 62.0. Temp. of water in system averaged 61°F. with variation from 48° to 80°. System contained 1,845,190′ of distribution mains of which 34,826′ were laid in yr.; transmission mains totalled 128,258′; pump mains, 10,422′. 1513 fire hydrants are located in Pasadena, 145 in county, and 24 in San Marine. System contains 3799 gate valves and 974 flush tanks. Meters total 27,109, tabulation being given by kinds and sizes. Total services, 32,543, made up of 27,006 meter

services, 103 irrigation, 88 fire services and 5,346 sealed services. Water consumption, ave. 12.5 m.g.d., with ave. per capita use of 117.18 gal. on basis of 3.6 persons per service. Normal total hardness of water from mixed sources supplying city is 147 p.p.m.; alkalinity varies in different sources from 96 to 203 p.p.m., total hardness from 85 to 218 p.p.m., chloride from 7.5 to 28 p.p.m., pH from 7.56 to 8.32. Report includes a number of tables containing comparative data in considerable detail. Bottom of outside cover contains important statistics of year's operation.—*Martin E. Flentje.*

Tientsin, N. China, British Municipal Council, Report for '36. 28 pp. Compiled by F. W. G. CLARK, W. W. Eng., covering operation of the municipal water works supplying water to British, French and German Concessions. Original filtered river supply has been entirely supplanted by water from 8 artesian wells. 100% well water being used since '34. In full commission, wells have a total cap of 2 m.g.d. Peak demand for '36 was 1,692,200 gal.; total sales for year, 363,558,860 gal.; an increase from the '35 figures of respectively 1,538,300 gal. and 351,089,580 gal. (3.55%). 50 lbs. normal pressure is maintained in dist. system. 7,798′ of new mains were laid, 16 new section valves installed, 19 A.G. hydrants and 1 steamer hydrant installed. Distribution system now contains 146,267′ of 3″–10″ pipe, 283 valves and 293 hydrants. 199 new metered services made in year bringing total in use to 2773. Income $327,872.87, expenditures $322,759.84. Discussion is given of future supply problem due to presence of fluorine in the well water accompanied by sodium salts that interfere with making of tea, causing complaints from "discriminating tea drinkers." Report contains plan of distribution system, balance sheet containing both notations in Chinese, and various tabulated results and figures of previous years.—*Martin E. Flentje.*

Rand, South Africa, Water Board, Annual Report. Year ending Mar. 31, '37. 104 pp. To the Minister of Public Health, Union of S. Africa. Water is supplied to Johannesburg, 8 other districts and mines with total population of 1,139,688 within a total area of 439.9 sq. mi. Sources of supply consist of Vaal River, boreholds in Klip River Valley and wells. Total water raised from all sources was 12,342,482,000 gal.; 83.71% of this coming from the Board's Vaal River res., total water sold was 11,932,807,000 gal. Ave. consumption, 32.69 m.g.d. (previous yr. ave. 27.53 m.g.d.) with min. of 22.65 and max. of 43.77 m.g.d.; highest ave. monthly consumption Mar. '37, 36.98 m.g.d. Rainfall for year was 40.57″ compared to 34.26″ of previous yr. and 48 yr. ave. of 32.79″. System consists of 367 mi. of pipelines made up of 343 mi. of pumping and gravity lines and 24 of well and borehole collecting lines. Total hardness of supply varied from below 60 to over 110 p.p.m., hardness being below 60 p.p.m. 27 days; from 60 to 70, 53 days; from 70 to 80, 113 days; from 80 to 90, 111 days; over 90 remainder. During year 177 chemical and 623 bacteriological examinations were made. Dosages of alum used were, ave. 1.27 gns. per gallon, max. 2.96 min. 0.75; lime, ave. 7.25, max. 10.1 and min. 4.3 gns. per g. Ave. cost of pumping and distribution amounted to 6.0 pence which included provision for contributions to Reserve and Betterment Funds at rate of 1.51 and 0.24 pence per 1000 gal. Total unit cost amounted to 10.34 pence per 1000

gal. as compared to 11.39 pence per 1000 gal. for preceding year. Report contains graph of water sold yearly since 1905–06, and large number of tables.—*Martin E. Flentje.*

Report of the Water Board of New Bedford, Mass., for the year 1936. ANON. The 67th annual report of this board serving New Bedford, Pop. 109,898; Acushnet, pop. 3,400; Dartmouth, pop. 7,300; total 120,668 of which 119,868 receive water. Publication contains short report of Water Board, detailed report of Water Registrar, Alfred Bradley, and detailed report of Stephen H. Taylor, Supt. Annual income, 1936, from water amounted to $349,689.17. An outlay of $144,732.69 for relining an old 48″ steel force main brought total outlay for construction of works to $6,911,412.64, exclusive of debt, interest, management and repairs. Cost of supplying water figured on (a) total maintenance less credits was $50.64 per m.g., (b) total maintenance plus interest on bonds was $66.28 per m.g., (c) total maintenance plus interest plus bond payments—$94.02; average rate received per m.g. pumped amounted to $90.08, and $109.06 per m.g. metered. Rainfall for year at pump station was 54.13″—9″ greater than average for past 42 yrs. Small size distribution pipe in system (less than 4″) equals 7,822′. System contains 213.45 mi. of pipe with average size of 13.89″. Total pipe laid in 1936—0.06 mi. No. fire hydrants in use—1903. Of 19,400 total services 17,365 are metered, 250 are unmetered fire services, and 1777 are not in use. Water consumption varied in 1936 from min. of 6.232 m.g.d. to 15.481 m.g.d. with a yearly daily average of 9.753 m.g.—an increase in the average figure of 0.308 m.g.d. or 3¼%. Daily per capita consumption was 79 gal. and gal. per tap 446. 83.3% of all water pumped was accounted for. Chemical character of water is given by tables listing reports of Mass. Health Dept., water is soft (8–14 p.p.m. total hardness) and slightly colored (10–40). Booklet also contains numerous tables, among them: water debt, comparison of 1936 consumption with record of previous 55 yrs., data on metered consumption and revenue, daily rainfall records and comparison by months with past 42 yrs. statements of work done, duty, etc. of steam and electric driven pumps, summary of kinds and length of pipe and sizes in system, list of makes and sizes of meters, and a 6 p. summary of statistics in form recommended by New England W. W. Assoc.—*Martin E. Flentje.*

The Water Supplies of Swedish Communities. ALFRED JERDEN. Gas-u. Wasser., Special annual number, 80: 673 ('37). Most communities having a population of over 1,000 have a municipally owned water supply. As only small local postglacial sand and gravel deposits exist the occurrence of ground water is limited and irregular. Yet, as most communities are small it is possible to supply about one-half of them with ground water. Where treatment of these supplies is needed it is mainly necessary to remove iron and manganese. Surface water is plentiful even if the average rainfall is only about 20″. The water of the rivers and the many lakes is soft and of low bacterial content and can be filtered in ordinary sand filters at rates up to 5,200,000 gal. per acre per day. Where the water is colored by humus materials coagulants are used followed by sedimentation and rapid sand filters. The water consumption has increased rapidly, in 1910 it averaged 28.6 gal. per capita in the 10 largest

cities. In 1935 it was 47.6 gal. per capita. The introduction of water meters tends to reduce the rate of the increase in consumption. All water works provide an adequate fire supply by using large mains, elevated tanks and many hydrants.

A short description of the water works of the three largest cities is given. *Stockholm*, with a population of 544,000, has obtained its water since 1933 from a new plant on the island Lovön in the lake Mälar. This plant has a capacity of 126 m.g.d., and an addition of 185 m.g.d. capacity is under construction. Precipitation with alum and rapid sand filters followed by chlorination are used. Taste and odor control causes some difficulty and the new plant will also have slow sand filters. *Gotenburg*, with a population of 263,000, has two plants. One plant uses lake water, which after slow sand filtration passes a marble filter to reduce the CO_2 content. The other plant uses river water. The greater part of this water passes through rapid sand filters, followed by slow sand filtration. About 14% of the output is treated with alum ahead of the rapid sand filters. About 2 m.g.d. are pumped into an artificial pond in gravel and recovered by well points about 300′ from the the pond after natural sand filtration. Aeration of this water is required to remove H_2S. Special precautions had to be taken to avoid the pollution of the river by salt water from the nearby sea. *Malmö*, with a population of 144,000, has had a ground water supply since 1901. Aeration followed by slow or rapid sand filters is used to remove iron and manganese.—*Max Suter.*

A Brief History of the Crooksville Water Supply Works and Water Supplies. C. T. Kaiser. Ann. Rept. Ohio Conf. Water Purif., '36, 82. The purification plant provided for the original well supply, which contained 9 p.p.m. of iron, consisted of an aerator, roughing filter and sand filters. Shortly after being placed in commission, iron content of well water increased to 70 p.p.m and plant was remodeled to provide for lime-soda treatment. As result of drought, well supply became inadequate, necessitating use of Jonathan Creek, flow of which is largely coal-mine drainage, water containing 50–100 p.p.m. of iron and acidity (sulfuric acid) as high as 1,500 p.p.m. Lime-soda was applied to neutralize acidity but softening was not attempted. In 1935-6 an impounding reservoir was constructed on small stream of relatively soft water (100 p.p.m. hardness), which is delivered by gravity to filter plant, eliminating low service pumping. Pipe line is made up of 1 mile each of steel, transite and c.i. pipe.—*R. E. Thompson.*

Water Supplies of the Province of Nova Scotia. R. D. McKay. Eng. Cont. Rec., **50**: 85; 17 ('37). There are 40 water supplies in Nova Scotia, serving 43 municipalities, 35 being derived from surface sources. Only 8 supplies are treated, 2 filtered and 7 chlorinated. The Dept. of Health's supervision includes 3 visits each year and examination of samples once a month. Samples are inoculated into bile broth (5 portions of 5 cc. each) at point of collection and examined for *Esch. coli* on arrival at laboratory. The province has been remarkably free from water-borne disease. Contamination of water in one system was traced to defective valve in raw water line direct to filtered water pumps which had not been severed when filters were installed.—*R. E. Thompson.*

Rehabilitation of Old Water Works Systems. D. H. FLEMING. Can. Engr., 73: 16; 5 ('37). Water works which have been in operation for many years often need to be rebuilt to fit them for continued economical service. Frequently, savings in annual cost of maintenance and operation effected by such rehabilitation are sufficient to finance the new works and in other cases the value of the improvement in quality, quantity or pressure is much greater than the actual cost. In *Port Stanley, Ont.*, the original supply, derived from infiltration basins on shore of Lake Erie, was most unsatisfactory and addition of 2 pressure filters and auxiliary intake did not effect much improvement. In '35, new 12″ c.i. intake, extending 1,600′ into lake and terminating in 20′ of water, was constructed and infiltration basins were converted into spiral-flow mixing chambers and settling tanks. Raw water is treated with alum and chlorine is introduced prior to pressure filtration. These changes, which cost $21,000, have corrected the difficulties and it has not been found necessary to increase the water rates. Intake, which has semi-flexible, rubber-gasketed joints, is protected from wave action at its mouth by a cairn of field stone, which, it is thought, will suppress force of waves and provide better key into lake bottom than would rigid, rectangular crib or masonry structure. At *Petrolia, Ont.*, water works consisted of intake at Brights' Grove on Lake Huron, pumps, 13-mi. supply main to town and 2 standpipes. Troubles included insufficient capacity and pressure and periodic turbidity. Remedies adopted were recaulking and strengthening of intake pipe joints with Dresser repair clamps, placing of bags of sand and cement around intake piers where they had been undermined and installation of low-lift pump, with suction connected directly to intake, alum mixing tanks and a settling basin. Latter was constructed in one end of old pumping station. Filters were not included as the water of Lake Huron is normally clear: periodic turbidities consist of relatively heavy particles of mud and sand, organic and trade waste pollution being absent. Quality of water, pressure and operating conditions have been materially improved.—*R. E. Thompson.*

New Procedures for Preparation of Drinking Water from Sea Water. RUDOLF BLAUM. Technische Blätter 27: 24, 362 (Jun. '37). Operation of sea water evaporators suffers from scaling of heating coils and mineralization of distillate by heavy carry-over due to sensitiveness for steam pressure fluctuations. These troubles are overcome at temperatures below 128°C. with carefully operated regulators, scaling being eliminated for periods of 3-4 months. On repeated trips of a Dutch passenger vessel distillate with 2-3 p.p.m. total solids was obtained for use as boiler feed and drinking water supply. In order to secure complete palatability and wholesomeness distillate is filtered through activated carbon and aerated. Estimated operating costs for supplying distillate for drinking, kitchen and bath demand are less than fresh water rates in harbors.—*Manz.*

Seawater for Sanitation in Virgin Islands City. ANON. Eng. News-Rec., 119: 684 ('37). Extension of salt water system for sanitation and fire protection in Charlotte Amalie, Virgin Islands, was recently completed by PWA. Seawater is pumped to concrete tank 250′ above town and delivered through network of underground pipes. Separate salt water system for sanitation and

fire fighting is almost unknown on mainland of United States but is necessary on Virgin Islands as average annual rainfall of about 45″ is insufficient to provide enough water from fresh water wells to allow ordinary sanitary improvements. Mains and sewers, both of c.i. pipe, were lined with cement to prevent corrosion, which entails constant replacement of unlined pipes.—*R. E. Thompson.*

Buenos Aires Loses Water in Offshore Wind. ANON. Eng. News-Rec., 119: 727 ('37). High offshore wind on Oct. 9, blowing waters of River Platte so far toward Uruguayan shore, 35 mi. away, that city's water supply intakes were left high and dry, caused interruption of supply for 24 hrs, necessitating bringing water in sprinkling carts from lakes in city parks. In ship channel, where there is ordinarily 21′ of water, depth was reduced by wind to 6′.—*R. E. Thompson.*

ADMINISTRATION, PERSONNEL AND PUBLIC RELATIONS

Winter-Time Protection of a Water Works System. W. E. MACDONALD. Eng. Cont. Rec., 50: 93; 7 (37). Operating practices evolved in Ottawa to insure efficiency of water works system during extreme sub-zero weather are reviewed. Removal of snow from city streets increases difficulties in distribution system. The Ottawa River, the source of supply, contains frazil ice in late fall. In view of this, purification and pumping plant on Lemieux Island was erected immediately adjacent to specially-constructed sedimentation basin equipped with 2 intake ports. Summer port is located in north wall paralleling the current and winter port is situated in east wall at right angles to current and facing downstream toward the bay. Planned to install 2 sets of traveling screens in winter port and equip them for application of steam as they emerge from water. Roofs of settling basins are insulated with cork board but, notwithstanding this, ice forms on surface of water in extremely cold weather, necessitating emptying to flush ice down the 54″ drain before thickness reaches dangerous proportions. Two 51″ steel supply mains cross the river from island to mainland on concrete bridge structure. These are insulated with three 1″ layers of hair felt, covered with heavy roofing felt, and as additional precaution heating units are maintained in bridge during extreme temperatures. New 48″ steel cylinder reinforced concrete pipe line installed in 1936, supported across river on separate steel structure, was insulated with two 1½″ layers of rock cork, applied over expanded metal, with joints staggered, and in air space between pipe and expanded metal a steam pipe was placed to enable heat to be applied when necessary. Over the rock cork is ½″ coating of weatherproof plastic asbestos cement. Formerly, all mains were laid with earth cover of 5′–6″, but as frost has been found to penetrate now to depths in excess of 7′–6″ latter has been adopted as minimum cover for new mains. When frost penetration approaches depth of mains, pressure tests are carried out on fire hydrants and, if necessary, heat is introduced into mains by thawing machines to melt ice formations. Ice in mains disrupts lime coating on inside walls, forcing some or all of it into the water. In view of this, practice has been adopted of considerably reducing lime dosage during periods that ice formation may be expected. This has eliminated excess lime tastes. Hydrant laterals

and fire services are insulated with hair felt and mineral wool covered with ready roofing and sealed with pitch to make them waterproof, which is essential. Wherever possible, service connections are tapped into hydrant laterals to promote circulation. All hydrants are inspected in October and thereafter throughout winter hydrants in business section are examined daily and others twice each week. Water works truck equipped with steam boiler constantly kept under pressure responds to all fire calls in winter months. Department maintains fleet of 5 portable electric thawing machines: during average winter, approximately 900 services, 30 mains, 2,000 hydrants and 100 valves are thawed out. Average time for thawing ordinary domestic service varies from 3 to 10 minutes and average unit cost is $1.78. These machines may also be used in conjunction with induction coil to locate mains and services.—*R. E. Thompson.*

Preparing the Water Works for Winter. ANON. Pub. Wks., 68: 9: 38 ('37). Preparation of waterworks structures against freezing weather may be divided: (1) preparing automatic devices to do work required under freezing conditions, (2) putting in service those items that are only expected to function during winter, (3) doing work now that is needed and which would be difficult under winter conditions, (4) getting materials and equipment on hand for emergency use. In fire hydrants attention should be paid to the drip to be sure it works and water drains away quickly; condition of all valves should be checked; intake screens and spillways of reservoirs should be cleaned of leaves and trash; if means are at hand to combat frazil ice conditions these should be gone over and placed in operating condition. Chlorine stations should be kept about 50°F., overhead power lines should be inspected to see that poles will withstand ice and snow loads. Tanks may be saved from ice damage by floating logs scattered over surface. Provision should be made for thawing services, mains and hydrants. These and number of other suggestions are given in this condensed valuable one page article.—*Martin E. Flentje.*

Why Water Works Associations—How Can They Best Serve? HARRY U. FULLER. W. W. and Sew. 84: 326 ('37). Water Works Associations must educate. They must split their meetings into local assemblies, make it easier for operators to attend.—*H. E. Hudson, Jr.*

Silvicultural Aids in the Control of the Gypsy Moth in Watershed Forests. A. C. CLINE. J. N. E. W. W. A., 51: 235 ('37). Control of the gypsy moth, an important shade and forest tree pest of the Northeast lies largely in the cultivation of forests having less than 50% of those trees whose foliage is favored by moth caterpillars. Oaks, gray birch, and poplar are among the most important and common favored food species, others are apple, alder, basswood, willow and hawthorn. Dispersion of gypsy moth is by air carriage of caterpillars when extremely small, vehicular traffic, shipment of nursery stock and in the short migrations of the larger caterpillars in search of food. Newly hatched caterpillars cannot feed on pine or spruce, these trees are only fed upon after caterpillar becomes about ½" long. Suggestions are given for control in existing forests.—*Martin E. Flentje.*

New Procedure Under the Public Health Act of 1936. S. SMITH. Surveyor (Br.) 92: 279 (Sep. 3, '37). Part IV, of the new Act, relating to water supply, contains an important change of procedure in that there is no distinction as between urban and rural authorities, as was the case under the Public Health (Water) Act, of 1878.—*H. E. Babbitt.*

Missouri Epidemic Damage Award Appealed. ANON. Eng. News-Rec., 119: 727 ('37). Springfield City Water Co., Springfield, Mo., has appealed to Springfield court of appeals a judgment rendered last month by circuit court at Ozark in which a consumer was awarded $2,999 on his claim that he contracted typhoid fever in summer of '36 from drinking water supplied by company. Case is first to be tried of 77 suits (involving damages of about $400,000) pending against company which arose out of dysentery and typhoid epidemic during July, '36. In that month, an estimated 25,000 cases of dysentery developed in city. Analyses of city water showed presence of gas formers (confirmatory tests not carried out) on July 15 and *Esch. coli* on July 18, 19 and 20. On July 23, further tests were inaugurated and during next 3–4 days paratyphoid-like organisms were found. About 10–14 days later, typhoid began to develop in city, rising to about 240 cases.—*R. E. Thompson.*

IMPOUNDING RESERVOIRS

Burnhope Reservoir Opened. ANON. Civ. Eng. (Br.) 32: 373 (Oct. '37). The reservoir, when full, covers an area of 103 acres and contains, 1,357,000,000 gal. (Imp.) of water. The project embraced: the reservoir, a public road diversion, eight lines of catch waters, and a tram road which was subsequently altered to an access road. The bellmouth weir at the outlet is 50′ in dia. at the top, tapering to a 12′ diameter shaft in a depth of 25′. The cost was £850,000; being £100,000 below the estimate.—*H. E. Babbitt.*

Silting of Reservoirs. ANON. Eng. (Br.) 144: 233 (Aug. 27, '37). Experiments show that if a turbid water is admitted to the head of a model reservoir a cloud of suspended matter progresses more or less in close contact with the bed until the weir is approached. It then makes its way along the upstream face of the dam to pass over the crest. Under sluices, if their sills are placed at a sufficiently low level, will act advantageously toward keeping down silting. All the silt does not pass in this manner, however; many factors affecting it. Besides, the design and operation of under sluices present problems of magnitude which may be minimized by preventing measures to decrease silt, such as erosion control.—*H. E. Babbitt.*

Pond Bottoms Sealed by Bentonite. C. C. DAVIS. Eng. News-Rec., 119: 725 ('37). Brief outline of procedure employed to correct bottom seepage on 3 ponds at Norris fish hatchery, each about 2.1 acres in area. Material used was grade No. 80 KWK Volclay, mined and processed at Belle Fourche, S. D. Ponds were drained, allowed to dry, plowed to depth of 8″, rolled 3 times with sheep's-foot roller, dragged to eliminate irregularities, treated with the binder, which was raked into top 1.5″ of soil, rolled again until feet of roller no longer sank into bottom and then dragged once more to smooth out the surface.

Neglecting evaporation, which is much higher percentage of total loss after treatment than before, losses before and after treatment, respectively, in g.p.m. were: pond No. 1, 350: 56; pond No. 2, 270: 32; pond No. 3, 167: 17.—*R. E. Thompson.*

DAMS

Hydrometer Computations in Soil Studies Simplified. EDWARD E. BAUER. Eng. News-Rec. **118: 662** ('37). Discussion of factors involved in determining percentages of various sizes of soil particles smaller than No. 200 sieve by hydrometer method, in which suspension is made and its density determined at definite time intervals, Stoke's formula being employed to compute diameter of particles. From series of readings from 30 seconds to 24 hours after agitation it is possible to calculate data for range of particle sizes between 0.0015 and 0.065 mm. To reduce amount of computation necessary, B. presents tables for the portions of the formulas which are functions of known physical data.—*R. E. Thompson.*

Measuring Soil Moisture. D. O. EHRENBURG. Eng. News-Rec. **118: 708** ('37). Portable electrical instrument, called a moisture-meter, for determining moisture content of sand and soil is described. In addition to the moisture content, conductivity of sand and soil is also affected by degree of compaction, temperature, composition and grading, and mineral content of contained water. Compaction can be controlled by employing standard procedure. Temperature-correction system has been developed which makes unnecessary calibration of instrument at several temperatures. Composition and grading is most important and limits employment of moisture-meter to the laboratory. Salt content of water is of little or no importance if clean tap water is used. (Cf. Wuerpel, Eng. News-Rec., July 11, '35.)—*R. E. Thompson.*

The Fernilee Reservoir. ANON. Civ. Eng. (Br.) **32: 334** (Sep. '37). The reservoir, located at Stockport, covers 86 acres and has capacity of 1,095,000,000 gal. (Imp.) The dam, which has a maximum height of 132′, is of the earthfill type with a puddle core wall, continued into the rock as a concrete cut-off wall. The volume of earth fill is 350,350 cu. yd.; excavation 28,542 cu. yd.; and concrete 23,880 cu. yd. A 12′ dia. tunnel, 735′ long takes water from the reservoir. The Stockport aqueduct consists of 30″ c.i. pipes 7¼ mi. long.—*H. E. Babbitt.*

Impounding the River Derwent. ANON. Civ. Eng. (Br.) **32: 336** (Sep. '37). The reservoir will be formed by an earth dam, 1,250′ long and 141′ high; containing about 900,000 cu. yd. Duplicate, bellmouth overflow weirs will be provided. The rim of each overflow is 80′ dia. and curves into a 15′ dia. horizontal tunnel.—*H. E. Babbitt.*

The Failure of Gravity Dams. ANON. The Engr. (Br.) **44: 215** (Aug. 20, '37) and **44: 242** (Aug. 27, '37). Tests were made upon the shearing strengths of various kinds of masonry blocks and cements representing an element cut

from the toe of a dam. The models were made to represent a height of dam h by varying the vertical load acting on the area A, and maintaining other factors constant. Results of the tests have been formulated into the expression

$$N = \frac{\beta S_o}{h\rho_w} + \frac{N}{\beta}$$

in which N is the minimum factor of safety against sliding; S_o is a value of S, the ultimate shearing strength of the joint, when the maximum normal stress is zero; h is the height of the dam; N is the coefficient of friction; β is the ratio of base width to height of dam; and ρ_w is the unit weight of water. It is evident, with everything else constant, N varies with h. It is thus possible to find a value of N for corresponding values of h for any assumed conditions.—*H. E. Babbitt.*

New Masonry Dam for Torquay Corporation. ANON. Civ. Eng. (Br.) 32: 359 (Oct. '37). The dam is a straight, masonry structure of the gravity type. The body of the dam will contain 46,000 cu. yd. of cyclopean masonry. The outlet tunnel will be D-shaped, 10′ high and 12′ wide. During construction the river will be diverted through a temporary tunnel in the spillway section. The contract of the construction of the dam was for £147,000.—*H. E. Babbitt.*

Comparison of Ruby Dam Designs. LARS JORGENSEN. Eng. News-Rec., 119: 633 ('37). Comparative estimates for straight gravity and constant-angle arch structures were made for Ruby Dam site, on which will be built a dam to create additional storage on Skagit R. for City of Seattle, Washington. For fixed amount of useful storage, i.e., 3,000,000 acre-feet, saving on arch type was estimated at about $5,000,000. Contract was let in August for first step in construction of constant-angle arch dam which will bring structure to height of 200′; second step, to be constructed later, will raise it to 400′ and ultimate height will be 625′. Initial contract provides for base width adequate for 400′ height. It is pointed out that there has never been a failure of an arch dam, whereas 69 gravity dams have failed to date, mostly by sliding. There are almost one-third as many high arch dams in existence as there are gravity dams of comparable height. See following abstract.—*R. E. Thompson*

Not a Constant-Angle Dam. W. J. MCKEEN. Eng. News-Rec., 119: 774 ('37). Cf. previous abstract. It is pointed out that structure now under construction at Ruby Dam site is not a constant-angle arch dam, as stated by Jorgensen. Ruby Dam is to be built ultimately to over-all height of 640′. Because of dense population of Skagit Valley below dam, City of Seattle has insisted that safety be given first consideration in design of the dam. Details of ultimate design are not yet complete, but it is expected that it will be approximately that used for Boulder Dam.—*R. E. Thompson.*

Rain Wrecks Spillway of Louisiana Dam. ANON. Eng. News-Rec., 119: 765 ('37). Spillway of Valentine Lake Dam in Kisatchie National Forest near Alexandria, La., was washed out down to creek bed level after unusually heavy

rain of 5.35″ during 24 hrs. ending 7 a.m., Oct. 3. Lake created by dam was completely emptied. Rammed earth dam was essentially undamaged. Spillway was lined with creosoted timber planking.—*R. E. Thompson.*

Bad Rock Limits T. V. A. Dam Location. ANON. Eng. News-Rec., 119: 665 ('37). Notwithstanding handicap of one of the most unsatisfactory sites on which a great masonry dam has ever been located, excellent progress has been made on construction of Chickamauga Dam across Tennessee R. 7 mi. above Chattanooga. Project is part of general scheme for development of Tennessee R. for flood control, navigation and power generation. Site is underlain with badly fissured and cavernous limestone. The conditions governing selection of location are reviewed. Much exploratory work was carried out. In order to crowd dam downstream as far as possible to secure better foundation conditions, the rather unusual expedient of including long embankments angling upstream from the abutments on both sides was adopted.—*R. E. Thompson.*

Elements of Chickamauga Dam. L. G. WARREN. Eng. News-Rec., 119: 667 ('37). Dam consists of concrete structures across main river channel, including navigation-lock, spillway and powerhouse sections, flanked by earth embankments across valley on either side. Total length is 5,800′. Embankments are being constructed of rolled clay soils: south one is 2,870′ long and has maximum height above original surface of 56′: north one is 1,410′ long and rises to maximum height of 50′. Concrete gravity-type spillway which crosses natural channel of river is 960′ long, deck having average height above rock of 111′ and maximum height of about 120′. Eighteen gates, 40′ X 40′ between piers 8′ thick will control flow of stream. Pool created will extend upstream some 59 mi. area at normal headwater elevation being 32,000 acres and capacity 541,000 acre-feet.—*R. E. Thompson.*

Bids Asked for High Dam at Grand Coulee Project. ANON. Eng. News-Rec., 119: 767,772 ('37). Bur. of Reclamation has asked bids on completion of Grand Coulee high dam, foundation structures for which are now being built under contract expected to be completed in Jan. '38. Award of new contract will be made soon enough, it is hoped, to permit work to proceed without interruption. New contract will provide for completion to full height of 553′; lower dam contract will bring height to 177′. Also included in contract is construction of power plant on west bank and protecting dam for pumping plant which will raise water from Grand Coulee Dam reservoir to balancing reservoir in the coulee itself for irrigation of Columbia Basin lands. New contract will involve placing of about 5,800,000 cu. yds. of concrete, bringing total concrete yardage to 10,300,000, 158,000,000 lbs. of steel and 340,000 cu. yds. of excavation. Dam will have crest length of 4,140′. Overflow spillway in central portion will be 1,650′ long and will be controlled by 11 floating-drum gates, each 135′ long and 28′ high. Spillway will include 30 pairs of outlet conduits to regulate reservoir during normal operation. Each conduit will be controlled by high-pressure gate 8½′ in diameter.—*R. E. Thompson.*

TUNNELS AND AQUEDUCTS

Ground Subsides on 2-Mile Line. ANON. Eng. News-Rec. 119: 136 ('37). Ground surface subsidence along 2-mile length of Colorado Aqueduct distribution system some 35 miles east of Los Angeles has been observed as it has progressed during 3.5-year period. In that time, maximum difference in levels has been slightly less than 1 foot, recorded just northeast of La Verne. In this area, aqueduct distribution line consists of pre-cast concrete pipe, 11.5′ in dia., with steel lock-joints calked with lead gaskets. As aqueduct will be under pressure, no reduction in carrying capacity will result from continued settlement and it is believed that flexibility of joints will accommodate settlement. In La Verne, foundation of well drilled to depth of almost 1,000′ in 1928 was found to have settled away from pump, leaving casing protruding about 3″. Casing was cut off 2″ inside foundation. Alva Smith, city engineer of La Verne, believes subsidence is result of lowering of ground water, level of which has dropped 100′ in 10 years.—*R. E. Thompson.*

Surface and Subsurface Investigations, Quabbin Dams and Aqueduct. A Symposium. Trans. A. S. C. E. 102: 679 ('37). **Foreword.** F. E. WINSOR. Page 680. The main features of the $65,000,000 project are the Quabbin Aqueduct, a tunnel 24.6 mi. long, with a 127-sq. ft. waterway, and the Quabbin Reservoir, in the valley of the Swift River, with a capacity of about 1,270,000 acre-feet. The Ware River is diverted into the Quabbin Aqueduct through a 260′ vertical shaft. Thus, water may be diverted either to the Quabbin Reservoir or to the Wachusett Reservoir. **Permeability Determinations, Quabbin Dams.** S. M. DORE. Page 682. The permeability of the overburden at each site was determined by two general methods: (1) The classification of bore samples and the estimation of the permeability of each class from laboratory analyses, and (2) The use of the pumping rates, the resulting ground water levels, and other pertinent data in connection with the lowering of the ground water at each site by pumping to determine the average effective permeable quality of over-burden as a whole. **Geologic Features, Quabbin Aqueduct.** F. E. FAHLQUIST. Page 712. The rocks of the tunnel area can be divided into two major classes, schist and granite. Data of a geologic nature are correlated with engineering and construction problems. Difficulties that might have been encountered were eliminated. Cost data in the several rock formations are tabulated.—*H. E. Babbitt.*

Oxyacetylene Welded Pipe Joints—Scope, Utility and Economics. T. W. GREENE. Metal Progress, 32: 36 ('37). Many piping systems of different metals are constantly being produced by oxyacetylene welding. Principal limitations are the properties of pipe itself. New back-handed technique for steel, with multi-flame blowpipes, excess acetylene and low alloy filler rods, reduces time to one-third and improves the quality.—*R. E. Thompson.*

Colorado River Pipeline. ANON. Eng. News-Rec., 119: 730 ('37). Extensive tabulations are given of unit prices from 3 low tenders on various sections of Palos Verde feeder of Colorado River aqueduct, involving 17.4 mi. of pipe line. Alternative bids were invited on 50″ and 54″ precast concrete

pipe, 42″ centrifugally-cast concrete pipe, 51″ and 55″ unlined welded steel pipe with Gunite exterior coating, 53″ and 57″ Gunite-coated welded steel pipe with coal tar enamel lining, and 51″ and 55″ Gunite-coated welded steel pipe with spun mortar lining and Gunited joints.—*R. E. Thompson.*

Some Recent Installations of Continuous Stave Wood Pipe. E. J. COUSENS. Can. Engr., 73: 18; 7 ('37). Much of leakage from wood stave pipe lines in past has been due to standard practice of using thin steel tongue to seal abutting ends of staves. In addition, due to attraction of metal to frost, ice formed at joints causes trouble. The Kelsey malleable iron butt joint, while effective from standpoint of leak prevention, aggravates ice troubles owing to its all metal construction. The raised metal ribs on inside of pipe is an objectionable feature of this type of joint and cost is high. A few years ago, a patented double mortise and tenon all-wood joint was developed which eliminates most of difficulties. This joint is proving to be highly satisfactory even under severe winter conditions. Recently constructed pipe lines on which the joint was employed are described briefly. In one case, joint was also used in construction of wood stave surge tank, 12′ in diameter and 70′ high, installed in connection with new pipe line.—*R. E. Thompson.*

Applications of Metallic Arc Welding are Increasing in Number and Importance. C. R. WHITTEMORE. Eng. Cont. Rec., 50: 98; 9 ('37). Factors which contribute to sound welds of good physical properties are selection of correct type of electrode for material to be welded, design of joint and proper welding technique. Most outstanding metallurgical contribution has been replacement of "bare" wire electrode with electrodes covered with mineral coatings having slag-forming characteristics, metal deposited being characterized by higher ductility, tensile strength, impact, fatigue and corrosion resistance. Only positive method of ascertaining degree of homogeneity of deposited weld metal and completeness of fusion throughout weld section is by means of radiographic inspection, using either X- or Gamma-rays. Former has proved more satisfactory. New 48″ supply main for Toronto, Ontario, is being fabricated in 12′, 18′ and 30′ lengths of $\frac{1}{2}$″ and $\frac{9}{16}$″ steel plate, rolled and automatically welded by the Unionmelt process. Welded steel shell is tested hydraulically, wire mesh reinforcing is tacked inside and pipe is centrifugally lined with $1\frac{1}{2}$″ of concrete. Pipe of this type embodies ability to withstand internal and external pressures, minimum coefficient of friction, minimum leakage and permanency of construction.—*R. E. Thompson.*

Bids on First 3 Sections of New York Water Tunnel. ANON. Eng. News-Rec., 119: 771 ('37). Bids were opened on November 9th for first 3 sections of Delaware River aqueduct tunnel, aggregating 101,100′ in length. Low bids on 3 sections totaled nearly $34,000,000. All 3 sections involve 2 shafts, each 310′ to 530′ deep. Finished diameter is 15′, minimum thickness of lining being 10″.—*R. E. Thompson.*

Arc Welding Continually Advances in Pipe Line Construction. ALTON F. DAVIS. Metal Progress, 32: 46 ('37). Examples are given to show recent

commercial developments. High quality of results, together with simplicity, control and economy of operation, indicate increased uses.—*R. E. Thompson.*

CONCRETE

Behavior of Concrete with and without Protective Coatings in Swamps. VICTOR RODT. Zement. 25: 601 ('36). Specimens made of 3 classes of cements were stored partially immersed in swamp waters. After 2.5 years, appreciable loss of lime from cement was observed, with increase in calcium carbonate content. Protective coatings of gas tar, asphalts, etc., were found intact above and below, but not at water line. All exposed concretes were appreciably etched after 7 years because of solution of cement.—*R. E. Thompson.*

Reservoir Seepage Traced to Earthquake. ANON. Eng. News-Rec., 119: 513 ('37). Complaints by residents in neighborhood of San Pedro Reservoir in Los Angeles, that water was seeping into their basements led to discovery of crack in bottom extending full length of reservoir. Test holes showed ground to be saturated. It was concluded that crack probably resulted from 1933 earthquake. Concrete was not reinforced and there were many smaller cracks. Major crack and all others of serious proportions were cut out and filled with concrete and bottom of reservoir was then paved with 2″ of flexible asphalt concrete containing only fine aggregate. Wall cracks were repaired by Aquatite process. Tests after refilling showed small leakage.—*R. E. Thompson.*

Steel Stresses in Concrete Pipe. D. B. GUMENSKY AND N. D. WHITMAN. Eng. News-Rec., 119: 597 ('37). In connection with design of large reinforced concrete pipe for Colorado River aqueduct and distribution system of Metr. Water Dist. of Southern California, series of tests was made in which actual strains in reinforcing steel were measured while pipe was subjected to combined external and internal loads. Although tests were made on pipe of same diameter (60″), the various sections were constructed and reinforced in different ways. Following conclusions have been drawn from analysis of data obtained: (1) Reinforced concrete pipe of commercial dimensions may be designed by formulas for reinforced concrete members under combined bending and direct stresses. (2) Concrete pipe reinforced with a steel cylinder remains watertight to ultimate failure. (3) For pipes reinforced with thin steel cylinder, highest strength for given amount of reinforcement is afforded by inner cylinder, outer circular bars and elliptical bars. (4) Bond between steel cylinder and concrete in pipe fails under relatively low bending stresses. Steel cylinder has uncertain value in resisting bending, but takes its proportional part of direct tension under internal hydrostatic pressure. (5) Concrete pipe reinforced with elliptical bars only has limited resistance against hydrostatic pressure. However, external vertical loads delay cracking and extend to somewhat higher heads the limit of its usefulness. Under hydrostatic pressure alone the pipe fails by longitudinal cracking of concrete on outside of shell along minor axis of steel ellipse. (6) Strength and action of

centrifugally spun pipe is definitely superior to pipe poured in vertical forms standing on end.—*R. E. Thompson.*

WELLS AND GROUND WATER

Hydro-Power for Pumping Developed by the Spokane Water Department. ALEX LINDSAY. W. Cons. News, 12: 142 ('37). Description is given of Spokane's new water supply and hydro-electric power development involving a reinforced concrete spillway dam and a new 5400 h.p. power plant costing $1,184,000. Water supply consists of large-diameter wells sunk in water-bearing gravel in extremely large underground flow. Drawdown with 5 wells in one field in operation producing about 60 m.g.d. does not exceed 3′. New wells and stations to be north and south of present Up-river plant. Description is given of design and construction details.—*Martin E. Flentje.*

The Question of Protective Zones in the Hygiene of Drinking Water Supplies. KNORR. Gas-u. Wasser., 80: pp. 330, 350 ('37). Proves that wells and springs should be protected against pollution by protective zones. Many examples are given showing that the size and shape of such zones depends mainly on geological conditions and on the use of the land.—*Max Suter.*

The Construction of the New Ground Water Supply for the City of Riga. RICHARD PAVELS. Gas-u. Wasser., Special annual number, 80: 677, 716 ('37). The original water supply of Riga was from the river Dūna, but after severe epidemics of typhoid a ground water supply was inaugurated in 1904. In 1929–32 careful studies were made to find additional ground water. Thiem's method was mainly used to determine the yield and the probable interference of the wells. These studies showed the district with the most ground water, its direction of flow and indicated that the wells should be spaced 300′ apart. In the construction the wells were spaced 600′ apart with arrangements for sinking the intermediate wells later on. In all, 29 wells were sunk in a line about 3 miles long at right angles to the direction of the flow of the ground water. The wells are 100′ to 150′ deep and gravel-packed with two layers of differently sized material, the outer layer being of about $\frac{1}{16}''$ in diameter, the inner from $\frac{1}{8}''$ to $\frac{3}{8}''$. Many details of this interesting well construction are not given but it is stated that the gravel-packing with two layers is successful and reduced the time for clearing the well water by pumping about 60%. The water is pumped from the wells by siphoning. The conduit from the well groups has its highest elevation in a collecting well, where the vacuum is applied. The suction line to the pumps, which are located in a deep pit in the pumping station, starts in the collecting well. Two pumps, one at 2400 g.p.m. with 200′ head, and one at 4400 g.p.m. with 230′ head, are provided, as well as space for a later additional pump. At present the pumps have runners to pump only 1600 and 2800 g.p.m. respectively, but the electric motors are of the size for use at the larger delivery. The power is obtained from the city power plant and a reserve Diesel power unit of 525 h.p. is provided. Two vacuum pumps, of 250 cu. ft. per min. capacity each, maintain the vacuum on the siphon lines. The pumps can be put in operation from this plant or from the

old pumping station, and are also connected automatically to the water level in the main storage tank in the city. The cast iron pressure line to the city is 4.5 miles long and has a diameter of 32″.—*Max Suter.*

Experience with Wells in Fine Sand. E. T. ARCHER. Eng. News-Rec., 119: 471 ('37). Ten years' experience with well system of Amarillo, Texas, shows that problem of obtaining continuous supply of water from fine sand and silt formation has been solved successfully. System, without addition to water-producing facilities, has increased in capacity in proportion to enlarged demands, population having increased from 25,000 to 50,000. City is located in semi-arid region: rainfall is so light that surface supplies are out of question. Present supply, lake-and-well system located 16.5 miles from city in valley of Palo Duro Canyon, consists of series of 10 wells constructed around artificial lake formed by 42′ dam across canyon. Tests showed depth of 175′ to Jurassic beds, lower 80′, which is tapped by wells, consisting of very fine sand (passing 50–100 meshes to inch and averaging 80). For first 4–5 years no water accumulated in reservoir. Open screens were used in construction of wells through which fine sand is drawn, sand ejected being replaced with gravel. Development was by system of air injections followed by air-lift and backblowing. Ten carloads of gravel, ¼″–1¼″ and averaging ¾″, were required for each well. At present time, wells are backblown every 6 months, ½ yard of gravel being required to replace fine sand extracted. Tests at time of construction showed that in some instances gravel had traveled as much as 28′ from well. When first placed in service, capacity of wells was 160 g.p.m. this increased with subsequent backblowing and replacement of fine sand and at present they will deliver 700 g.p.m. each. Air-lift is used in 8 wells and centrifugal pumps in 2. Water passes through sand traps to small storage reservoir, from which it is pumped by centrifugal pumps against 100′ head through 30″ pipe line to reservoir in city. Plant is operated by electricity purchased at 1.81¢ per kilowatt hour. Average cost per 1,000 gal. during 5 months of past year was: production 1.626¢; transmission 0.969¢; distribution 1.030¢; total 3.625¢. Consumer rates of 12–27¢ per 1,000 gal., provide ample revenue. Capacity of wells of ordinary type developed 4 years later 2.5 miles from city receded rapidly from initial capacity of 5 m.g.d., failing entirely in 4–5 months. By permitting them to stand idle for several months the operation can be repeated, but initial capacity is less each time, maximum being now only about 50% of original capacity and operation can continue for only 2 months.—*R. E. Thompson.*

Administrative Control of Underground Water: Physical and Legal Aspects. H. CONKLING. Trans. A. S. C. E. 102: ('37). The use of underground water in the United States is increasing continually. In humid regions there are few statutory enactments as to rights to use of water but in arid states administrative control has been established. In California the land owner is permitted reasonable use of water but in Colorado severe restrictions on the use of underground water exist. There is some confusion between legal pronouncements concerning the use of surface water and the use of underground water. The water authorities of the arid states generally grant a permit to an

applicant for the use of surface waters. In applying an intelligent administrative control it must be recognized that the reuse of underground water increases the dissolved solids content; materially affecting the usefulness of the water. In certain states, Idaho in particular, pumping which lowers the water table for some prior user is prohibited. As control of underground water becomes necessary the complexities of the situation can be dealt with only by lodging greater power in the water authority than is now given in most jurisdictions.—*H. E. Babbitt.*

PUMPING

The Oswestry Booster Plant. F. H. AUGER. Trans. Liverpool Eng. Soc. 58: 13 ('37). The Vyrnwy Aqueduct, 68 mi. long with a total fall of 548′, supplies Liverpool with water from Lake Vyrnwy in Wales. Water shed has a maximum capacity of 48 m.g.d. The aqueduct was designed for an ultimate capacity of 54 m.g.d., to be carried by four 42″ pipes laid in parallel. Two complete lines supplied the necessary capacity until '36. The conduit system has four balancing reservoirs along its length, thereby breaking the lines into five distinct sections. Short lengths, approximately 5 mi. each, of a third line were laid in '26, in each of the five sections and interconnected to the two complete lines. These portions of the third pipe increased the capacity enough to last until '32. At Oswestry, there is a large reservoir on the conduit, then a drop of 118′ in less than one-half mile to the Oswestry Filters. Water flows by gravity from the filters through the remainder of the conduit to Liverpool. This excess head at this point was formerly dissipated by throttling at the filter inlets. When another increment of conduit capacity became necessary in '32, the available power in this 118′ drop was utilized to postpone the addition of more conduit capacity by boosting the effluent from the filters. Careful study and design was necessary because the volume of power water and pumped water would always be equal. This equality was modified by taking a constant flow of 2 m.g.d. to operate an electric generating unit. This unit is equipped with an electric brake which maintains a constant load on the water wheel. The electric brake automatically follows the variation in the connected electrical load of the generator. The hydraulic turbines and pumps are direct-connected horizontal shaft units. The turbines have Francis impellers and are equipped with oil relay governors, whose most important function is to prevent runaway speeds in case of a break in the pump discharge line. The net head available at the turbines is about 94′. The unit also has a by-pass around the turbines, with an automatically controlled valve of a type similar to the Larner Johnson valve. This valve automatically bypasses water in excess of that required for power under partial load conditions. Two units have been installed and a future third unit will complete the layout. Overall efficiencies of 64.6 to 64.9% were obtained from the first unit. 70.0 to 71.8% efficiency was obtained from the second unit. The conduits leading away from the plant have check valves in the gravity line from the filters, located above the junction of the pump discharge. In case the pumps are not running, the filters continue to supply the conduits by gravity. The installation of the two units, with all housing and connections cost about $161,000. The estimated cost of additional conduit capacity to accomplish the same

purpose was $877,000. The future need of the fourth conduit has been eliminated.—*Homer Rupard.*

The Springwells Station of the Detroit Department of Water Supply. W. C. RUDD AND B. J. MULLIN. Trans. A. S. M. E., **59**: 4; 297 ('37). Following a description of the basic plant design and layout, authors discuss engineering studies made in connection with the design and construction of the plant, with especial reference to hydraulic and mechanical features of the pumping plant and mechanical and electrical characteristics of the steam operated power plant. Purchase of pumping and power plant equipment secured on competitive bidding, with some of the major items secured on evaluated bids and with bonuses and penalties applied to contractors' guarantees is described in detail. Results of field tests are outlined, with the resulting bonuses and penalties applied to the various contracts. General plant operating economies for a one-year period are discussed.—*Homer Rupard.*

DISTRIBUTION—MAINS, VALVES, TANKS AND RESERVOIRS

The Discovery of the Losses of Water. H. DALLDORF. Gas-u. Wasser. **80**: 426 ('37). Loss is considered as any unaccounted for water pumped into the distribution system and total loss as any difference relative to the raw water pumped.

The losses are subdivided into

A. Losses of unused water (leakage, overflow).

B. Water not measured (flow too small for meter, incorrect estimates on fire and sprinkler use).

C. Apparent losses (inaccuracies of master meters).

Methods for the determination of the different losses are given: "A" losses can be found by listening at night or direct inspection of house connections. "B" losses can be reduced by regularly exchanging the meters in use for two years against newly tested ones or by the installation of recording meters. "C" losses require a regular check on the master meter. A statistical method for checking water losses is also given, using graphical presentations to show the trend in the distribution of the losses.—*Max Suter.*

A Remarkable Reservoir. ANON. The Eng. (Br.) **164**: 263 (Sep. 3, '37). The new reservoir for the city of Nantes, France, is three-storied, with a capacity of 8.8 mil. gal. (Imp). It is so constructed that the stories can be independently filled and emptied. The structure consists of a series of thin arched sections, in reinforced concrete, resting on beams and buttresses.—*H. E. Babbitt.*

The New Large Storage Tank for the Water Works of the City of Vienna. RUDOLF TILLMAN. Gas-u. Wasser., Special annual number, **80**: 701. This reinforced concrete tank is of 38 mil. gal. capacity. It is built on a hill near the end of the second conduit bringing the spring water to Vienna. It will help to equalize peak flows and allow shut downs up to four days for repairs of the main conduit from the springs. The tank is of the flat slab type of construction and covered with 20″ of soil. Owing to topographical conditions

it is of two unequal compartments, one holding about 22 mil. gal., the other about 6 mil. gal. The water depth is 20′. The structural details and the control of the concrete were handled carefully to allow high stresses in the material. To avoid shrinkage stresses temporary openings were left in the slabs and walls, and to take care of unequal settling in the footings a special type of permanent joint was used. The tank does not float on the line, as the water is forced to flow through it. A special pressure system provides for circulation in dead corners.—*Max Suter.*

New Standard Specification for Fire Hydrants. ANON. Contract Jour. (Br.), p. 778 (Sep. 15, '37). The British Standards Institution has recently issued a British Standard Specification relating to underground (flush) fire hydrants and the dimensions of the surface-box openings. The specification among other things provides sufficient clearance between the outlet and the operating stem, and a uniform outlet to fit a standard adaptor.—*P. S. Wilson.*

Water Tower Fails in Yonkers. ANON Eng. News-Rec., 119: 697 ('37). A 120′ brick masonry water tower supporting a 285,000-gal. steel water tank on hill in Yonkers, N. Y., fell on Oct. 23, releasing flood of water and crumbled masonry which caused damage to nearby houses and injured 6 persons. Structure, known as Nodine Hill Tower, was part of city water works system and served the high pressure lines. Foundation consisted of 3′ brick masonry wall based on a rock outcropping. Lower 40′ of tower, which had inside diameter of 28′, consisted of 2′ brick wall: next 40 feet was 18″ thick and final 40 feet was 15″ thick. For 8′ farther, 15″ brick wall surrounded the 20′ steel tank. Application for PWA project to demolish tower and construct new one had been approved but no money was allotted. WPA project for repair of roof of tower was under way at time of failure but no work was actually in progress when accident occurred. Tower was built in 1891. Pending investigation, officials are of opinion that failure was in tank rather than in tower, and that rush of water caused destruction of latter.—*R. E. Thompson.*

DISTRIBUTION—METERS AND SERVICES

Copper Alloys for Thin-Walled Pipes. A. S. BALL. Tsvetnye Metal., '37, No. 2; 87. Alloy of copper containing 1.5–2% manganese is very strong and resists corrosion as well as or better than other copper alloys.—*R. E. Thompson.*

Service Maintenance Truck with Air Compressor. H. S. MORSE. Eng. News-Rec., 119: 362 ('37). Brief description of new service maintenance truck, placed in operation by Indianapolis Water Co., designed to carry all equipment necessary for maintenance and replacement of customer services from main to property line, including air-cooled compressor of sufficient capacity to run 2 paving breakers.—*R. E. Thompson.*

Sizes of Meters for Services. R. W. ESTY. J. N. E. W. W. A., 51: 245 ('37). Meter sense, developed through experience, aids in selecting proper size meter for water service. In author's opinion a laundry or irrigation line requires a velocity type meter due to intermittent use of large quantities of

water; a school building, shoe factory etc. should have a disc meter because demand is small and uniform; leather factory or lamp factory should have compound meter. Based on size and service, estimate of consumer and manager's experience, a meter of theoretical size should be set in service and then checked with recording device with either 6 or 24 hr. chart records of water being used.—*Martin E. Flentje.*

Noise in Plumbing. How to Prevent It. PIERRE KULA. Technique Sanitaire (Fr.), **32**: 4: 79 ('37). Noise must be fought on two fronts; (1) its production, and (2) its transmission. It arises (*a*) in the supply lines (where trouble is greatest), (*b*) in waste lines, or (*c*) in fixtures. *Supply Lines.* A supply system of lead pipe of uniform bore, without sharp elbows or constrictions and terminating in a swan-neck dipping under water, would be noiseless under pressures far exceeding those customary. But should turbulence, shock, or vibration, be set up at any point, whether in pipe, or at tap, or in water itself, noise throughout the system will follow and be transmitted to the building itself. In lead pipe, even with sharp elbows, noise in the pipe itself is generally negligible; but in copper pipe, it is noticeable, and in iron pipe, still more so. Change of internal diameter, or abrupt change of direction at a branch may cause murmuring noises audible over the whole system and even transmitted to masonry. Accessories of all kinds, faucets, stop-valves, flush-valves, float-valves, check-valves, meters, and pumps cause much more serious noises. Author considers that makers of valves and taps have still much to learn in the matter of silent functioning. Noise at any given tap will increase enormously with increased pressure. It is therefore of primary importance to know the minimum pressure necessary to yield normal flow, and to establish that pressure. Author gives as norms for the most exacting requirements, 3½ g.p.m. for wash-basins, sinks, etc. and 6½ g.p.m. for baths, which means that with both taps open only three minutes are needed to prepare the bath. With good taps of ⅜″ bore for the baths and of ½″ bore for the basins, etc., a 5′ head of water is ample for these flows. Author regards therefore as axiomatic that (1) noise can be greatly reduced, but not completely eliminated, (2) noise hardly noticeable at low pressures will increase with pressure and may become intolerable at 60 pounds, and (3) when running wide open, very low pressures of order of 3 pounds at taps and of 12 pounds at flush-valves are adequate to maintain the flow. Valve-maker's task is to turn out valves which will give the least possible noise with the highest possible pressures. Plumber's task is to plan the installation so that the pressure at each tap will approximate that just necessary for normal flow. Hence the usual direct pressure system, where vertical risers from a basement circuit supply by successive branches the successive floors, is faulty in principle. Upper floor taps are the last to be supplied; they have lowest static head and greatest loss of head both by length of pipe and by users at lower levels; so that lowest taps gush noisily and uppermost ones are irregularly and inadequately supplied. When, to remedy this condition, the pipes are made larger, the upper floors, it is true, are benefited; but the pressure at the lowest taps is increased. Much to be preferred is a circuit at the topmost level with descending vertical feeders to the lower floors. In this system the uppermost taps are fed directly and the

increased static head on the lower floors is in part neutralized by the loss of head in the pipe and by users at intermediate levels. A further great improvement is to introduce a pressure-reducing valve on the main supply which will at all times maintain the pressure which is most desirable, namely that required to yield a normal flow at the uppermost taps, that is 15 pounds, or maybe something less. The descending mains should be only just large enough to provide adequate flows at periods of heavy demand. Anything larger is detrimental, because loss of head in pipe is a desirable check on increase of static head. It is not possible in practice to have the pressure in branch leading to each individual tap at its optimum value. Regard must always be had to periods of peak demand; to possible additions to number of fixtures; to furring up; and, in case of iron or copper pipe, to noisy clicking in the pipe when tap is turned off if bore of pipe be too small. Most taps must therefore of necessity be over-supplied, except at periods of peak demand. This difficulty is met by means of a pressure-reducer for each tap. This cannot be either a throttle-valve or a diaphragm, for these are themselves noisy. A suitable noiseless contrivance is a spiral of small bore pipe, the length and bore of which are chosen to meet the requirements of its particular case. Requirements, therefore, to be met by good systems include: (1) instantaneous normal flow simultaneously available at a suitable number of taps; (2) pressure-reducing element to prevent noise for each tap; and (3) taps of large bore, larger even than the branches supplying them, so as to ensure a full normal flow without noise. Very instructive diagram shows pressures which will prevail in vertical feeder mains at branch to each story of seven-storied building to supply a flush-valve, pressure in city main being 75 pounds, (*a*) under direct pressure system, (*b*) with overhead circuit, but without pressure-reducing valve, and (*c*) with overhead circuit and pressure-reducing valve. Taps which open with difficulty will sometimes cause a gushing noise because the pressure reducer is without effect when flow is small. Flush-valves, especially when bowls require a sudden large flow in order to flush properly, will require correspondingly increased pressures. Such noises cannot be avoided, but their transmission can, as will be seen. Stop-valves cause noises of much the same kinds as taps. They should be of good construction, without any part capable of vibrating, and with washers in good order. They should always be kept wide open and never used as pressure regulators. Divisional meters are also noisy; sometimes, intolerably so. Unless chosen sufficiently large, the runners may attain sufficient speed to set up a loud humming, which carries to a great distance. Makers of meters are inclined, in order to encourage sales, to offer meters of sizes barely sufficient for intended duty. It is easy, when testing a meter, to note the flow at which humming starts. This should never be exceeded. Pump runners behave similarly to meter runners; sound may even become shrill at high velocities. Transmission to piping should be prevented by insertion of short lengths of rubber or durite hose. Transmission to building is prevented by bedding pumps on rubber blocks. Check-valves often slam very disagreeably. This can sometimes be avoided by using high-class valves of sufficient diameter and judicious location; it is, however, better practice to omit check-valves entirely. Any fitting which closes under pressure, as for instance a check-valve, will cause water-hammer unless its closing

is in some way restrained. *Transmission of noise.* May take place by means of (1) the piping, (2) the water, (3) the masonry, or (4) the air. A simple form of silencer, effective for piping and water alike, is a short length of rubber or durite tube of suitable strength inserted in the main by means of Prache joints, and surrounded, with sufficient intervening space, by a copper sleeve which will take care of a possible failure of rubber. Although lead alone is not noisy, yet a lead insert is of no effect. The water itself is often the principal vehicle of noise transmission. Masonry takes up noise from pipe fasteners, or from embedded pipe. To correct, insulate pipe with swanskin ("molleton"), or similar material. This must be done with meticulous care, as a single defective point will destroy the effect. Experiments of Lyon are recalled, where an internal chamber was rigorously insulated by means of rubber blocks from an external surrounding one, and it was found that one single nail driven through from the outer wall to the inner one sufficed to destroy the insulation. If, however, the production of noise has been satisfactorily kept down, transmission through the masonry will be of secondary consequence. Nevertheless, sleeping apartments are better without piping of any kind, as far as possible. Where piping must pass through masonry, it is best grouped together into sleeves judiciously placed and constructed of insulating materials. Transmission to the air is best avoided by having a sufficient thickness of insulation on the pipes. Water-hammer has not been dwelt upon, because in no properly erected system will it develop. It is due either to defective fittings, or to excess pressures. Its destructive effect is more to be dreaded than the noise. If it cannot be eliminated, then air-chambers should be used to deaden it. Noise in the *waste system* occurs through (*a*) siphon discharge, or (*b*) the rush of water down the pipes. Noise from siphons is suppressed by the well-known device of individually ventilating each one to a common secondary ventilation network. The rushing noise is hardly to be avoided. Its transmission is to be guarded against by insulation and by locating these pipes remote from apartments where absolute quiet is essential. Noises in the *fixtures.* None of the above precautions avail to prevent the noise of filling of a basin or a bath, or of the flushing of a toilet-bowl, or of the aspiration which follows the emptying of a flushing tank, or of the metallic noises from the same. For a wash basin, the least noisy filling is through a mixing tap giving a full-sized, quiet, vertical cylindrical jet. A bath supply through a hose will be noiseless once the end of the hose is submerged. Toilet noises can be kept down by (1) bowls which do not need a violent flush and which do not aspirate noisily and (2) high-class flushing mechanisms, especially designed to give silent operation; by preference, either a low flushing tank of a flush-valve. If flushing tank is used, it is necessary that it should fill quietly, always remembering that hygiene forbids the submergence of the jet which delivers the water. Summing up: (1) it is possible to execute plumbing which will be almost noiseless; *but* (2) it can only be done by observing many precautions which must date right back to the preparation of the plans; it is necessary that the plumbing contractor be taken into consultation by the architect; in any other way, only palliatives can be applied and with only partial success. Neither is it to be forgotten by the architect that though the plumbing be noiseless, he has still the users to reckon with. The author instances one building where the entrance hall is

directly under a large lavatory of toilets and wash-basins. One can distinguish clearly from below sounds of walking, talking, and of opening and closing of doors, although never the slightest sound from any of the fixtures.—*Frank Hannan.*

HYDRAULICS—HYDRAULIC ENGINEERING

The Calculation of Pressure Losses in Water Pipe Lines. RUDOLF TRAUTE. Gas-u. Wasser., 80: 268, 285 ('37). Gives nomograms for the calculation of the pressure losses in metric system for different types of pipes and using different European formulas.—*Max Suter.*

Comparison of Sluice-Gate Discharge in Model and Prototype. F. W. BLAISDELL. Trans. A. S. C. E. 102: 544 ('37). Models of sluice gates can be depended upon to predict the discharge of their prototypes with reasonable accuracy. Froude's model law will apply to the discharge of sluice gates; roughness of both model and prototype must be given consideration in the construction of the model; and good results should not be expected for small gate openings and low velocities. In general, $Q_n = kQ_m l^{2.5}$ in which Q is the discharge, n denotes "prototype," m denotes "model," l equals scale ratio of length l_n/l_m, and k is a constant. The mean departure of Q_n/Q_m from $l^{2.5}$ was found to range from −4.6% to +4.5%, with an average departure of +0.4%.—*H. E. Babbitt.*

Crest Lengths Classify Discharge. R. ABBETT. Eng. News-Rec., 119: 594 ('37). Some of the variations in weir formulas can undoubtedly be attributed to changes in approach channel conditions, but on other hand, discharge coefficient in common theoretical formula has been found to vary with both crest length and head. If, instead of using theoretical value of 3/2 for exponent of head in common weir formula, correct exponent is determined, then discharge coefficient will be found to vary only with crest length. While plotting variation of discharge coefficient and exponent against crest length for 11 experiments on sharp-crested weirs an interesting fact was revealed. In each of 2 zones the exponent has constant value; in zone between other 2 it is variable. This condition is somewhat analogous to that existing in pipe flow where flow changes from laminar to turbulent at critical values of Reynolds number. In these experiments, crest lengths varied from 0.5″ to 48″, heads ranged from zero to 1′, and weirs were sufficiently high to render influence of approach channel negligible. Interesting side light of these experiments is the nappe's ceasing to cling to crest at practically same head (0.08′ to 0.10′) for all weirs. Results of experiments on each weir were expressed by formula of type $Q = KLH^n$, and discharge coefficient, K, and exponent, n, for each weir plotted against crest length, L. Study of results indicates that crest lengths between 0.5″ and 2″ have constant value of 1.50 for exponent, n, while crest lengths greater than 8″ have somewhat lower constant value of 1.465. Separating these 2 "constant value" zones is transition zone in which exponents' value varies from 1.465 to 1.50. A universal weir formula therefore seems impossible. Each weir in transition zone behaves according to its own individual law and results of tests on such weirs are certain to be mislead-

ing if applied to weirs of different crest length. Majority of weirs in practical engineering work have crest lengths exceeding 8″ and thus fall in upper "constant value" zone. Flow over these can be expressed by empirical equation $Q = KLH^{1.465}$, where discharge coefficient K is itself an exponential function of crest length. This equation is not dimensionally correct unless K is also function of H. In this case, dimensionally correct equation is written $Q = C\left(\frac{L}{H}\right)^{0.035} LH^{3/2}$. It is seen therefore, that any attempt to derive empirical weir formula with constant discharge coefficient will be restricted necessarily to one specific crest length and such a formula applied by direct proportionality to any other length than the specific one on which it is based will result in error.—*R. E. Thompson.*

Modern Conceptions of the Mechanics of Fluid Turbulence. H. Rouse. Trans. A. S. C. E. **102**: 463 ('37). The paper is a summary of efforts to solve the problems of resistance in turbulent flow using the flow in circular pipes as means of illustration. After a brief discussion of pipe resistance in terms of general dimensionless parameters, the physical nature of turbulence is described qualitatively; the inner mechanism of flow is then followed from laminar stage through the critical zone and well up into the region of high Reynolds numbers for both smooth and rough pipes. The Prandt-von Karman theory of the mixing length, the significance of the laminar boundary layer with relation to surface roughness, the latest experiments on artificial roughness, and von Karman's universal relationship between the velocity distribution and resistance to flow, are finally treated in terms of the foregoing discussion of the mechanics of fluid motion.—*H. E. Babbitt.*

On the Transport of Dissolved Material by Flowing Liquids. Sverre Stene. Gas-u. Wasser. **80**: 534 ('37). Mathematical formulas are developed showing the function of the concentration of a dissolved material in a liquid flowing through one or more basins. The general differential equations are given and solved for several cases, such as sudden or continuous introduction of the dissolved material in one or more basins, uniform sizes of the basins or simple relations of their sizes. The theoretical results are applied to the test by dyes in the determination of the possibility of pollution of wells. They are also used in the discussion of the limits and errors of Thiem's method using salt for the determination of the lineal velocity of ground water and to Aastad and Sögnen's method for the measurement of the flow of water in rivers.—*Max Suter.*

Sedimentation in Quiescent and Turbulent Basins. J. J. Slade. Trans. A. S. C. E. **102**: 289 ('37).

$$B_r = t_r(D_{r+1} - D_r) \tag{1}$$

$$B_a = \sum_{r=1}^{s} B_r\left(1 - \frac{a}{k_r t_r}\right) + \sum_{r=s+1}^{n}\left(B_r e^{-\frac{a}{t_r}}\right) + B_c \tag{2}$$

in which B_r = quantity of suspended solids with an average hydraulic value V_r; t_r = time of subsidence corresponding to V_r; D_r = derivative of the expression $\sum_{r=1}^{n} B_r(1 - a/t_r) + B_c$ in which $r = 1, 2, 3, \ldots n$, successively; B_a = quantity remaining in suspension at the end of time a; S = a number of parts less than n; a = time during which sedimentation occurs; k_r = a coefficient. $k_r t_r$ produces average time of subsidence under certain conditions of turbulence for particles with time of subsidence in a still basin equal to t_r; B_c = quantity in a colloidal state, including that part of the settleable solids with hydraulic values V, so small that for practical purposes they may be considered as in colloidal suspension. Equation (1) gives the relation between the distribution of hydraulic properties of the sediment and its settling characteristics in a quiet tank. The distribution of hydraulic values of a sludge may be determined by observing its settling behavior in a glass cylinder. Knowing this distribution, its settling characteristics in an actual tank may be predicted by equation (2). Before any practical application can be made it is necessary to determine t_i; t_c; k_r; and other constants, in which t is the time of subsidence, i as a subscript denoting incomplete, and c as a subscript denoting "colloidal," "complete" or "critical", as defined in each case.—*H. E. Babbitt.*

Exchange of Hydrotechnical Research Results. Bulletin No. 6(14) U. S. S. R. Comm. for Exch. of Hydr. Lab. Research Results. Leningrad. '36. **Study of Wave Action on Flat Walls,** p. 8. An empirical formula has been devised for the determination of wave loads upon walls. It depends upon the radio H/L, where H is the depth of the water and L is the length of the wave, and upon the angle of inclination of the wall. **Protection Against Silting of Intake Works of the Jaroslav Rubber-Asbestos Combinate, on the Volga River,** p. 20. The investigation was conducted upon a model representing at 1:75 scale the intake works and part of the Volga River channel. Directions of bottom and surface currents and transportation of bed load (sand) were observed. It was shown that designed intake structures are apt to be entirely silted up. To combat this phenomenon the method of artificial lateral circulation was used. A floating stream-guiding installation stopped the silting of bottom openings of the intake. Owing to wide fluctuations of the river, separate stream-guiding installations have been designed for high and low water periods. On the basis of the investigations a new design of intake structure was prepared, using the method of lateral circulation. **Investigation of Capacity of the Tuloma Spillway at Various Gate Openings and at Various Upper Pool Elevations,** p. 24. The coefficient of discharge of the spillway corresponding to one meter head was found to be 0.39; at 7.2 m. it is 0.44. **Investigation of Suitability of Adopted Outline of the Tuloma Spillway Face,** p. 25. The greatest vacuum produced is 0.6 m. A considerable vacuum has been detected downstream from the toe, on the horizontal floor. To eliminate that vacuum the shape and dimensions of the toe have been redesigned. **Determination of Size and Shape of Water Cushion and Energy Destroying Devices,** p. 25. Energy destroyers selected included the Rehbock indentated sill with some additional devices at the outlet. It has been found that a sill placed on the bottom of the stilling basin halves the bottom velocities in the basin, while

rows of dentated sills located at the outlet causes a redistribution of the velocities with a reduction of bottom velocities. **Study of Percolating Flow Through the Lower Tuloma Earth-Fill Dam,** p. 27. The upstream blanket is of little value in consequence of the infiltration of water into deep-lying, very permeable strata. A considerable residual head exists in the downstream part of the foundation, and there exists insufficient stability of the foundation against uplifting forces of the percolating water, in the region where the reverse filter is placed. **Study of Construction Methods to be Used for Closure of the River Channel by Rock-Fill,** p. 28. Studies were made on a three-dimensional model at 1:60 scale. The tests showed that a previously determined amount of rock was insufficient for building the fill up to the point where it shows above the water surface. The second scheme showed good results, on condition that certain precautions be taken against excessive percolation which may endanger the stability of the foundation. **Study of Percolation Through The Lower Tuloma Dam,** p. 32. As a result of the tests the profile of the dam was selected with an upstream blanket 140 m. long and with a horizontal impervious diaphragm 90 m. long. The tests have given interesting data throwing light on the rôle of the blanket in connection with the geologic conditions, its influence upon the flow net, the rôle of the diaphragm, etc. **Investigation of Dams on Weak Foundations,** p. 50. Some conclusions have been reached which may serve as a base in the design of timber low-head dams. Among other problems the rôle of lateral percolation and of sheet piles at the abutments was investigated.—*H. E. Babbitt.*

ERRATUM

Effect of Applying Pulverized Carbon Previous to Filtration. Discussion by R. N. Statham of paper by this title. Ann. Rept. Ohio Conf. Water Purif., '36, 79. *Editor's Note*: An abstract of the above discussion appeared in this JOURNAL **29**: 2074 (Dec. '37). We have been requested to correct certain inferences which have been drawn from this abstract. The original published statement from which the abstract was taken included the following:

"Although water with a threshold value of between 10 and 20 may not be objectionable under certain conditions since it is influenced by the type of pollution present, the method of determining the threshold point, and the sensitivity of the operator, the general consensus of opinion is that a threshold value of *more than* 3 will cause consumers' complaints. This standard of taste and odor reduction is being particularly emphasized by many of the large middle western cities where pollution of the raw water is an every day occurrence."

The words "more than" (italicized above) were omitted from the abstract.